GUIDELINES FOR THE DESIGN OF

WATER TREATMENT PLANTS

AND

SEWAGE TREATMENT PLANTS

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WATER

GUIDELINES FOR THE DESIGN

OF

WATER TREATMENT WORKS

APRIL 1982

THE HONOURABLE
JIM BRADLEY
MINISTER
R.M. MCLEOD
DEPUTY MINISTER

ACKNOWLEDGEMENTS

These design guidelines have been prepared with the assistance of the consulting firm of Simcoe Engineering Limited, Pickering, Ontario. The draft guidelines, prepared by the Consultant, underwent review by representatives of leading consulting engineering firms and various branches of the Ontario Ministry of the Environment.

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P R E F A C E

By issuing these guidelines, it is not the intention of the Ministry of the Environment to stifle innovation. Where the designer can show that alternate approaches can produce the desired results, such designs will be considered for approval.

The designer should note that the Ministry of the Environment has a number of policies which relate to water treatment systems, and which may affect their design. In all cases these policies take precedence over the design guidelines, and the designer should be aware of all current policies.

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SECTION 1

GUIDELINES FOR THE DESIGN OF WATER TREATMENT WORKS

1.0 GENERAL CONSIDERATIONS

1.1 INTRODUCTION

These guidelines have been prepared to document the desirable ranges, and the normal, minimum or maximum acceptable levels for the various design parameters used in the design of municipal water treatment plants requiring approval by the Ministry of the Environment.

A complete documentation of all parameters relating to water treatment plant design is, of course, beyond the scope of these guidelines, but an attempt has been made to touch upon the parameters of greatest importance from the process and reliability standpoints.

By issuing these guidelines, it is not the intention of the Ministry of the Environment to stifle innovation. Where the designer can show that alternate approaches can produce the desired results, such approaches will be considered for approval.

Wherever possible, designers are encouraged to use actual data derived from water treatment plant records, operational studies including pilot plant work, etc. rather than use arbitrary design parameters. This is particularly important with water treatment plant expansions where the designer

may want to use hydraulic and/or loading rates in the upper levels of the acceptable loading ranges, or where the designer proposes to deviate from recommended design parameters.

The mention in the following text of specific documents and reports is not intended to imply that these represent the sole, or most highly regarded sources of information. They may, however, be regarded as a starting point for the designer who may wish to use these documents, in concert with his own experience, to complete his design.

Designers are advised to familiarize themselves with the requirements of all legislation dealing with water treatment works, their associated equipment and labour safety requirements.

1.2 LEGISLATION AND APPROVALS

The Environmental Assessment Act, 1975 (EA Act) and the Ontario Water Resources Act, (OWR Act) are the two statutes administered by the Ministry of the Environment which have application to water treatment works.

The designer should refer to the Environmental Assessment Section, Environmental Approvals Branch of the Ministry of the Environment early in the design process to ascertain what requirements must be met under the Environmental Assessment Act, i.e. whether the Ministry Class Assessment applies, whether a hearing might be required, etc.

The general information required and procedures to follow in applying for approval under the Ontario Water Resources Act are outlined in MOE Publication, "A Guide for Applying for The Approval of Water Words". (1)

Projects financed and/or operated by MOE, and those where MOE provides project Management should be submitted to the Design and Equipment Section, Project Co-ordination Branch, for approval under the OWR Act. Projects financed by any other method, including upfront grants provided they are not managed and operated by MOE, should be submitted to the Municipal and Private Approvals Section, Environmental Approvals Branch, for approval under the OWR Act.

Any approvals required under the EA Act must be obtained prior to receiving approval under the OWR Act.

Water treatment works may be subject to other planning-oriented legislation, such as the Planning Act, The Municipal Act, The Ontario Municipal Board Act, and others. The Environmental Assessment Section of the Ministry of the Environment should be contacted for information on streamlining approvals under these various pieces of legislation.

In addition to these approvals it may be necessary to obtain approval from any number of other organizations which have jurisdiction over part or all of the project. Primarily, this will involve the Industrial Safety Branch of the Ministry of Labour, and approvals may also be necessary from public bodies

and authorities such as Ontario Hydro, municipal plumbing and/or building departments, conservation authorities, and the Federal Government (Parks Canada or the Department of Transportation) where intakes or outfalls are involved, and compliance with the Navigable Waters Protection Act is required. Liaison with utilities such as Bell Canada, Ontario Hydro, Gas companies, and CN/CP may also be required.

1.3 MINIMUM REQUIREMENTS

The minimum treatment requirements for potable water production are governed by policies of the Ministry of the Environment. The minimum treatment for groundwaters is disinfection; the minimum treatment for surface waters is chemically assisted coagulation floculation, filtration, and disinfection.

These minimum requirements only apply where such treatment will produce finished water from the proposed source of sufficient quality that it meets or exceeds the criteria outlined in the Ministry of the Environment's "Drinking Water Objectives". (2) In all other cases additional treatment may be required to meet these objectives.

SECTION 2

2.0 GENERAL DESIGN

2.1 GENERAL CONSIDERATIONS

Many processes other than those described later in these guidelines may be suitable for the production of potable water in special cases. Typical of these processes are slow sand or biological filtration treatments, reverse osmosis, ion exchange, activated carbon treatment, and flotation.

While these, and other processes, may be permitted, subject to adequate verification of their effectiveness, their applications are sufficiently limited that no further specific reference is included in these guidelines. The design guidelines following are related to the most commonly employed water treatment practices.

2.2 DESIGN CAPACITY

In general, the plant should be designed on the basis of projected flows for a 20-year period. For large treatment plants, or where construction cost is an overriding factor, a lesser design period may be selected, but it is recommended that the minimum design period be not less than 10 years. For intakes or outfalls, where the cost of the work is not as substantially dependent on the size used, a design period in excess of 20 years is suggested.

Depending on circumstances, including the reliability of projections, a design to satisfy the ultimate requirements of the official plan for the plant area under consideration may be appropriate.

The capacity of the plant is defined as the quantity of water which can be delivered to the distribution system when operating the plant under design conditions. This capacity should be sufficient to meet the maximum day demand, and greater capacities may be required depending on in-system fire flow requirements and storage capacity. In the absence of specific information relating to water consumption, the average daily rate and peaking factors discussed in the Ministry publication, "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems, and Storm Sewers" (3) should be used to determine the required treated water quantity to satisfy maximum day demand.

The treatment process capability must be greater than this quantity, since allowance must be made for water required for in-plant use for filter washing, service water, chlorine ejectors, etc. to be provided during the maximum day, as well as allowing for filter downtime during a complete wash cycle. Proper design and operation will normally require daily washwater quantities equal to 2% of design capacity for medium to large treatment plants, and up to 5% for small plants. The designer should be particularly careful in designing small treatment plants since in-plant usage can be a highly significant portion of total production.

The designer should also consider the capacity of the plant to ensure that it is possible to produce sufficient water to satisfy the most onerous combination of flow and raw water quality. Typically, this condition will arise in the spring when raw water

quality from surface sources is often substantially worse than average, when raw water temperatures are low (reaction times become extended and the efficiency of sedimentation tanks and filters is reduced) while the water quantity required may, in some cases, exceed the annual average flow (but not the maximum day). This will result in the occurrence of a "solids removal peak" and the component units of the treatment plant must be designed to accommodate this. Normally, the maximum day and solids removal peak requirements are non-coincident for most surface waters, although algae problems may worsen with increasing water temperatures and produce co-incident peaks.

2.3 SOURCE SELECTION

As a general guideline, the source selected should be capable of providing raw water which will meet the criteria described in Environment Canada Scientific Series #43 report, "Water Quality Criteria for Great Lakes Waters to be used as Municipal and Industrial Water Supplies". (4)

The designer should ensure that the water from the source selected is of sufficient quality that it can be treated without abnormal difficulties or costs to produce a finished water which complies with the Ministry's "Drinking Water Objectives".(2)

In cases where the designer has a choice between two alternate sources, he should investigate the pollution status and potential development status of each watershed, ensure that the watershed has sufficient safe yield to provide an adequate quantity of water,

and develop outline treatment processes and their associated costs prior to selecting a particular source.

2.4 SITE SELECTION CRITERIA

Some of the factors which should be considered when selecting a site for new treatment works, or the extension of an existing facility, include:

a. adequacy of isolation from residential areas or other non-compatible land uses which could be adversely affected by the plant;

b. optimum location of the plant with regard to the location of the raw water source and the area to be serviced. For example, a water pumping station at the source and a tment plant in a more accessible location ease of construction;

c. susceptibility of the site to flooding;

d. suitability of sub-surface and soil conditions;

e. adequacy of the site for future expansion;

f. minimizing adverse environmental impact both during construction and operation of the facility;

g. avoidance of construction adjacent to a shore line except where unavoidable, since suitable measures would otherwise be necessary to prevent erosion, and to protect structures from potential wave action or ice-piling.

2.5 PLANT LAYOUT

The general arrangement within the selected site should take into consideration the suitability of subsurface conditions to provide the necessary facilities at minimum cost. Where possible the designer should take advantage of natural grades in arranging the various process units. Consideration may be given to the use of inter-stage transfer pumps where they are more economical (capital and operating) than extensive construction in adverse ground such as rock.

In the layout of the plant the designer should locate the buildings to allow adequate flexibility for the economical expansion of the various treatment sections. Plant layout should consider making best advantage of prevailing wind and weather conditions to minimize energy consumption, e.g. locate units which need only moderate temperatures on northern exposures.

The plant layout should allow for the probability of snow drifting, and entrances and roadways should be located to minimize the effect of snow drifting on operations.

Roadways for chemical deliveries should be designed to be sufficient to withstand the largest anticipated delivery (typically a 27,000 litre tank truck), with due allowance made for vehicle turning and forward exit from the site. Roadways should be designed in accordance with A.A.S.H.O. WB-50 semi-trailer requirements.

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Within the constraints mentioned above, the designer should work towards a plant layout where the various processing units are arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit water from one module to the next, and also to arrange the plant layout to provide convenience of operation. Whenever possible, the process units should be located adjacent to each other to minimize the use of space and materials, and to minimize travel distances for maintenance crews.

The designer should note that some layouts are not acceptable; for example common wall or slab construction between finished water and raw water tanks. The plant layout should provide adequate protection for all finished or treated water units.

2.6 INTAKE LOCATION

The designer's objective in locating the intake should be to provide adequate quantities of raw water which will result in the lowest overall treatment cost, when both the plant and intake construction costs plus plant operating costs are considered. For example, further extending an intake a relatively short distance may produce raw water of much higher quality which can be more economically treated.

In determining the general location for an intake, historical aerial photographs may be of considerable value. Where possible, current depth soundings should be compared with historical data to ascertain whether particular locations are susceptible to silting or have large quantities of bottom sediment.

All available water quality information should be examined and the designer should take particular note of both present and future planned outfalls from sewage treatment plants and industrial installations, as well as any inshore pollution, especially during high run-off conditions. Data on current flows and directions should be reviewed, as well as potentially infrequent occurrences such as thermoclines or falling plume dispersions, to determine an intake location which will be free of adverse pollution.

The final intake location will be affected by water bed contours, subsoils, and available water depths, and the submerged depth will depend on the type of shipping, if any, which frequents the general location. The minimum submergence from top of intake structure to minimum recorded water level should be three metres wherever possible, and the preferred submergence is ten metres.

SECTION 3

3.0 PROCESS GENERAL

3.1 PROCESS DESIGN

The minimum treatment requirements for surface water are chemically assisted coagulation/flocculation, filtration and disinfection. In many cases, surface water requires "full treatment", i.e. chemically assisted coagulation/flocculation, sedimentation, filtration plus disinfection.

Where iron and manganese levels are within the Drinking Water Objectives limits and turbidity is below 5 N.T.U.; colour below 40 H.C.U.; and algae below 2 000 A.S.U., the designer may consider omitting the sedimentation process. This decision will be based on an economic evaluation of the process with and without sedimentation. As a norm, the designer should attempt to ensure a minimum filter run between backwashes of twenty-four hours at conventional filter rates.

In determining the particular treatment requirements for the raw water, it is recommended that the process design be based on operating data accumulated over a substantial period of time, i.e. at least three years, where the design is for an expansion to an existing plant. For the design of a new plant, pilot plant data are acceptable provided that such data have been accumulated over a significant period of time, and over a wide range of raw water conditions, similar to that which is historically demonstrated. Alternatively, the plant design may be based on operating data from a treatment plant obtaining water from the same source as the proposed plant, and which

is located nearby. Where no such data are available, it is a minimum requirement that treatability studies be conducted. Care should be exercised in extrapolating treatability studies since these are generally concerned only with the chemical treatment requirements, and will not provide design data for reaction and retention times, filtration rates, etc.

Where a direct filtration process is proposed, a pilot filtration study is required.

Treatability or pilot studies should be discussed with the Ministry prior to commencement, and ideally should be extended to cover at least one year.

Any sampling programmes conducted at the proposed intake location may be used to establish a data base for design.

The design of the process units should become progressively more conservative as the quantity and reliability of experimental or operating data decreases.

3.2 CORROSION CONTROL

The process design should be such that finished water is neither unduly corrosive, nor encrusting, and is chemically stable. In providing corrosion control it is recommended that a positive correction process be selected over one which provides only a protective coating to the distribution system.

Common control methods include the use of carbon dioxide, lime or other alkali. Alternative methods

of control will be approved if sufficient operating experience or data is provided.

Where carbon dioxide is used to provide a stable water, the contact chamber should provide a detention time to ensure complete reaction - typically fifteen minutes at design flow. When carbon dioxide is produced by combustion, adequate precautions must be taken to preclude the possibility of carbon monoxide entering any section of the plant.

Where lime or other alkali is used to increase the water pH, the lime system and application point selection should acknowledge that lime addition prior to filtration may result in carbonate precipitation in filters, or may resolubilize previously precipitated materials; i.e. high aluminum residuals in filtered water caused by floc dissolving at high pH values. Conversely, lime addition after filtration will result in increased turbidity due to inert particulate matter present in the lime, or deposition of sludge into finished water storage areas, or decrease the rate of disinfection and greatly increase the contact time required to effect the necessary disinfection.

3.3 DISINFECTION

While it is Ministry policy to provide finished water with a chlorine residual, disinfection of raw water may be accomplished by other methods. It should be noted that extensive experience of chlorination has demonstrated its safety and effectiveness to a greater degree than other disinfectants.

Alternate methods of disinfection which may be allowed include the use of ozone, potassium permanganate, iodine, ultraviolet irradiation and many others. The designer should give due consideration to the economy of the system proposed, the ease and simplicity of continuous operation, ease of analysis for determination of dosage and residual, and the state of knowledge concerning potentially hazardous by-products, e.g. epoxides formed by ozonation, and the effectiveness of disinfection. It is recommended that proposals to use disinfectants other than chlorine compounds be discussed with Ministry staff to obtain their recommendations early in the design process.

SECTION 4

4.0 RAW WATER SYSTEMS

4.1 INTAKE DESIGN

Because of the difficulty and high cost of marine construction, it is suggested that the intake size be sufficient for the projected plant requirement for an extended design period. This will often result in only a single size difference in the intake when compared to a 20-year design period. The hydraulic design for the intake should assume a Hazen-Williams coefficient of one hundred (C = 100).

The intake design and its anchoring should taken into account peak wave height and frequency, and provide adequate protection against ice scouring and dragging anchors.

The designer should obtain historical information on water depths at his proposed location, and determine whether or not the source level is regulated, and also whether historical minima occurred before or after such regulation.

The designer should refer specifically to the Ministry Research Report W42, "Occurrence of Frazil Ice in Intakes" for recommendations on crib design and inlet velocities. Intake crib materials should be of low thermal conductivity, with racks of "slippery" and light coloured materials. The design should provide for low entry velocities - below 75 mm/sec - and uniform acceleration of water from inlet to intake pipe.

Entrance ports to intakes should be located to prevent bottom sediments from being picked up. Both top entry and side entry designs are acceptable, and may be evaluated on the basis that side entry designs are less likely to be damaged by anchors, while top entry designs provide greater clearance above the river or lake bottom, and the required inlet area can be more readily obtained.

All designs should be checked for water hammer problems, particularly if the intake is long or design velocities high.

Intakes may need to be identified with buoys, and where they are in proximity to shipping lanes, radar reflectors may be necessary.

For small intakes, consideration should be given to providing means for back-flushing the intake, if practical.

Under certain circumstances, an intake may not be necessary, and a forebay may be constructed. The designer should ensure that sufficient depth of water will exist in the forebay under any source conditions, and measures should be taken in the design to ensure that the bay can remain relatively clear of ice.

The design of river intakes differs substantially from that for lakes, in that substantial currents may exist, and both anchoring and bottom scouring considerations will assume greater significance. River intakes should be located well upstream from potential sources of pollution.

4.2 SCREENING

Raw water screens should be provided to effect the removal of large solids, with the screen mesh size and materials of construction consistent with the raw water quality. In general, a screen mesh size of 10 mm is usual, and the screen should be sized for a maximum velocity of water through the screen of 0.6 m/s regardless of water level in the screen well. Where parallel screens are used, the maximum velocity with one screen out of service should not exceed 0.9 m/s.

Two screens should be provided; normally, two fixed screens in series for small plants, two rotating mechanical screens in parallel for large plants, and one rotating screen plus one fixed screen in series for medium capacity plants. In each case, the designer should ensure that sufficient space exists above the screen to permit removal.

Fixed screens should have suitable lifting lugs on the screen, and a lifting hook or beam positioned above the screen to assist in removing it. When it is intended that screens be cleaned manually, the designer should pay particular attention to the size of screen sections and materials of construction to ensure that screen removal and handling can be readily accomplished manually.

Rotating screens should have either automatic or manual advance mechanisms arranged so that complete washing of the screen is accomplished. The screen rotation should be such that even wear is obtained from all sections.

For either type of screen, washing facilities should be provided using high pressure water through an appropriately sized line. Provision should be made for diverting screen washings to a holding tank, preferably with a basket type screen, so that screen wastes can be simply dewatered prior to adequate disposal. Screen wastes may not be returned to the raw water well.

Screen wells should be watertight design with provision made either through valving or stop logs for isolation of the well for cleaning or inspection. The screen well should be covered, with a curb around any floor openings to prevent water running from the floor into the well. Wells should be provided with differential level indicating devices to permit the headloss across the screen to be determined to assess the need for cleaning.

4.3 RAW WATER WELLS

The design of the raw water well will be substantially dependent on the number, type, and size of the raw water pumps required. The design should conform to the recommendations of the Hydraulic Institute.

The well floor should be sloped to a sump for easy cleaning. The well should be covered, with a curb around any floor openings to prevent floor drainage entering the well, and adequately vented.

The raw water well should be provided with an overflow of sufficient size to handle intake surges which occur on power failure (when all pumps stop). Alternatively, the well above the water level should

be sufficiently high to contain this surge. When designing to handle intake surge, a minimum Hazen Williams coefficient of one hundred and fifty (C =150) should be applied.

In some cases, downsurge during hydraulic transients after power failure may cause problems if pump suctions become exposed and air becomes entrapped in the raw water pump suction piping.

4.4 RAW WATER PUMPING

Pumps should be specified so that the full range of flows anticipated can be provided with pumps operating in the vicinity of their optimum efficiency points, with due regard to the hydraulic design of the discharge piping. This is often accomplished by selecting pumps which have wide band efficiencies and a relatively flat operating curve.

The number of pumps should be consistent with the pattern of flow required and the method of flow control. It is recommended that at least three pumps be provided for operating flexibility; a minimum of two pumps is required, one as standby. Pump capacities should be such that with the largest unit out of service, the remainder will be able to supply the treatment plant capability (as required in Section 2.2).

Provision should be made for an individual pressure gauge on each pump, isolating valve and check valve on the discharge side, and a compound gauge on the suction side. dry well pumps should be provided with suction side valves. The designer should consider

the use of slow opening pump discharge valves at raw water pumping stations remote from the treatment plant. Piping should be arranged to allow ready disassembly from pump to shut off valves, and include a flexible type coupling to permit proper alignment of the piping and pump. Couplings must be adequately protected against thrust. Pump elbows should be supported to remove all bending moments, either steady or shock, from pump nozzles.

The station design should allow for future additional pumping units and where possible, the pipework should be large enough for an increase in pump size to be accommodated. Adequate space should be provided for the installation of these additional units, and to allow safe servicing of all equipment.

Minimum spacing around pumps should comply with the requirements of the Hydraulics Institute and the Ministry of Labour. Adequate space should be provided for removal of units, and in the case of vertical turbine pumps it may be necessary to provide a roof access for removing the units and sectional discharge pipes so that they can be completely removed from the raw water well.

All piping should be arranged so that there is sufficient room to service all valves and other parts, and to permit their removal with minimum disturbance to the system. A bridge crane, monorail, lifting hooks, hoist or other adequate facilities should be provided for servicing or removing equipment.

Water seals should not be supplied with water of a lesser sanitary quality than that being pumped, and where potable water is used for pump seals, protection against backsiphonage must be provided.

Pumps should be mounted on bases above the floor level, and all access openings into the well shall have suitable curbs around them to prevent floor drainage entering the well. The station floor should be sloped to floor drains. Floor drainage back to the raw water well is not permitted. Drainage from pumps on to the floor is not acceptable.

Care should be taken to ensure that unless the pumping station design provides for flooded suctions at all times, an adequate priming system is provided with sufficient capacity to prime pumps within a short period; i.e. 1 - 2 minutes.

Except in special circumstances where a plant is operated at a constant rate, an adequate control device should be provided which is capable of controlling the rate of flow from the raw water pumps over the entire range of flows to be treated. Where this is accomplished by control valves, it is normal to use butterfly valves operating in a range to maintain stable control and avoid cavitation. Other types of control system will be considered provided that they allow for stable flow control. At small treatment plants where substantial seasonal variations in flow exist, it may be necessary to provide duplicate flow control systems - one suitable and for very low flows (which normally occur in winter) and one suitable for the plant design flow.

SECTION 5

5.0 PRE-FILTRATION PROCESSES

5.1 COAGULATION - FLOCCULATION

The design objective of the rapid mix/coagulation step is to provide high intensity mixing to distribute the coagulant chemical uniformly throughout the raw water as rapidly as possible. Coagulation may be achieved either in a separate process tank or more usually by the use of an in-line mixing device. In either case, the minimum amount of energy should be consumed consistent with maximum performance of the rapid mix function.

Since the optimum rapid mixing can only be determined by pilot studies, a high degree of flexibility is required in the rapid mix design, except where it is based on pilot studies or related experience. Typically, energy gradients - G values - in the order of 1000 sec^{-1} are effective. The practice of adding coagulants in raw water wells and allowing pumping units to perform the mixing process is not recommended.

Following coagulation, flocculation is required to condition the suspended materials to suit the processes following, viz., sedimentation or filtration. The design of the flocculation system should be such that flocs formed are maintained throughout; i.e. low velocities are used, and rapid acceleration is avoided.

When sedimentation is required, typical flocculation times are in the order of 30 minutes at design flow, although longer times are required for cold waters,

and typical Gt values of 50000 - 125000 are required. Tapered flocculation - reducing G values for each stage - is desirable. Optimum G and Gt values may be determined by pilot studies.

When filtration without sedimentation is proposed, G and Gt values will be determined by the required pilot study. Typically, 15 minutes flocculation time is required.

To permit flexibility of operation and for maintenance purposes it is recommended as a minimum that two separate flocculation tanks be provided. To prevent short-circuiting during flocculation it is recommended that each tank be divided into at least two stages.

Both mechanically mixed and hydraulically mixed tanks are acceptable, provided that sufficient flexibility of operation is possible, and that G values can be varied to approach optimum flocculation.

Where mechanical agitation is provided, submerged bearings are not recommended, and all submerged parts should have sufficient corrosion resistance to withstand long term use with flocculated water and deposited sludge.

Tanks should be arranged to permit drainage and removal of deposited sludges for treatment.

5.2 SEDIMENTATION

The design objective of the sedimentation process is to produce water which can readily be filtered at the required capacity in the most economical manner. For example, a slight increase in sedimentation efficiency may allow much higher filtration rates and smaller filters at lower overall cost.

The minimum retention time should be one hour at design flow. Tank depth should be between 3.0 and 5.0 metres depending on tank size, plus an allowance for retained sludge.

The primary design parameter is the surface overflow rate which will vary according to the performance required in terms of clarified water suspended solids, the type of flocculated material generated prior to sedimentation, and the temperature of the water treated.

Typically, surface overflow rates for sedimentation tanks are from 1.6 - 2.4 m/h, and from 3.8 - 5.8 m/h for sedimentation tanks with tube settlers where the tube area considered is defined as the area of the receiving plane faces of the tubes, or the projected area of tubes perpendicular to the direction of flow.

Lower rates are normally associated with flocculated colour materials and higher rates associated with flocculated suspended material (turbidity).

Where water temperatures are consistently lower than 10° C, these rates should be reduced. For plant

capacities below 10,000 m^3/d, where sedimentation efficiencies are frequently lower than in larger plants, these rates may need to be reduced by 15 - 25% to achieve the desired results on a regular basis. If the flow is not evenly distributed to the receiving face and from the receiving face, these rates should be reduced.

Additionally, where tube settlers are used, a maximum underflow velocity of 1.0 m/min., based on tank cross-section, is recommended. The velocity across the top of tubes, from tube to weir, should not exceed 0.15 m/min.

The designer should allow for the possibility of ice formation within settling tanks which could fall and cause damage to submerged tubes or other components within the tank if the water level is dropped.

Inlets to settling tanks should be provided to prevent bottom scour at the inlet end, and to minimize short-circuiting. Maximum entrance velocities should not exceed 0.6 m/s. The outlet arrangement should draw water from the full width of the tank to prevent short-circuiting, either by a submerged pipe where velocities do not exceed 0.3 m/s, or across a weir where flow does not exceed 200 m^3/d per metre of weir.

Where it is necessary for a settling tank to be taken out of service for cleaning, a minimum of two settling tanks is recommended. Even where sludge is continuously removed from tanks, a minimum of two tanks is recommended. Where only one sedimentation tank is provided, sufficient finished water storage

must be provided to allow continuous water supply while the sedimentation tank is out of service. alternatively, provision should be made to provide adequately treated water without sedimentation during tank cleaning periods. It is recommended that a by-pass pipe or conduit be provided where practical.

Where it is proposed to remove sludge manually, the tank bottom should be sloped, typically 1 in 100, toward the inlet end. For sludge removal by travelling siphon or scraping mechanism the tank bottom should be flat. Where sludge is to be removed by pumping from sludge hoppers, the hopper design should be consistent with the flow characteristics of the sludge handled.

Sludge draw-off methods should take into account that sludge loadings near the tank inlet may be substantially higher than at other locations. Sludge withdrawal piping should be such that material withdrawn can be observed to ensure that sludge rather than settled water is being removed. Tank drainage should allow the tank to empty within a reasonable time, say 8 hours.

Suitable roof drainage should be provided, and such drainage is not permitted to discharge into the settling tank.

Where submerged outlets are used, each tank should be provided with a suitably sized overflow or other means to prevent flooding. Any overflow should be located so as to be readily visible.

All valve operators external to the tank which are not within buildings should be tamper proof design, with provision for locking.

Each tank should have vents and suitable personnel access openings which comply with the appropriate Ministry of Labour and other regulations. At least two openings are recommended to assist ventilation when work inside the tank is performed. The preferred locations are adjacent to walls so that permanently fixed ladders may be used to provide safe access. Openings into tanks should be curbed and covers should preferably have a locking device provided Additional small openings into the tanks may be appropriate for testing purposes such as dye tests for short circuit detection, or observation of settling characteristics.

For certain waters a pre-sedimentation tank in the form of an impoundment may be of considerable value during seasonal disturbances of the water source. Such impoundments may be constructed as embankment type lagoons, and if remote from the plant, should have suitable perimeter fencing.

A single impoundment with a by-pass is satisfactory, and in many cases some chemical treatment at the impoundment may be highly advantageous. The detention time and general design criteria for impoundments should be such that normal operation of the water treatment plant is possible even under extreme raw water quality conditions.

5.3 CLARIFIERS

As an alternative to the flocculation and sedimentation facilities described, proprietary clarifiers or combined flocculation/clarifiers may be approved if sufficient data are provided to verify their performance.

It is recommended that solids contact clarifiers be covered either by location within the plant or by the use of a separate dome type cover with personnel access to permit visual inspection of the treatment. Where open top clarifiers are proposed, the equipment should be properly weatherproofed and the rake mechanisms should be equipped with torque switches to prevent overloading. It should be noted that ice blockage of effluent launder orifices may occur unless these orifices are sufficiently covered to remain ice-free, thus increasing the operating depth of the clarifier.

An upflow rate equal to the design flow divided by the clarifier area excluding the central draft tube of 3.0 m/h is generally acceptable for Great Lakes waters, but where colder temperatures or difficult coagulation, e.g. colour, is anticipated this rate should be reduced. Where tubes are included, the rate may be increased to 6.0 m/h for Great Lakes water.

Recirculation impellers should have an adjustable speed range of 1 - 4. Rake speed should be variable from 0.3 - 4.0 m/min.

Where the proposed operation is "stop-start" mode, the design must allow sludge recirculation to continue when raw water flow stops, to prevent process upsets of sludge recirculation type clarifiers. The raw water inlet valve should be of the slow opening type operating over not less than one minute to prevent sludge upsets. Where operation may result in large rate of flow changes which tend to decrease effluent quality for short periods, declining rate filtration (see Section 6) may be more appropriate than constant rate filtration.

SECTION 6

6.0 FILTRATION

6.1 FILTRATION TYPES

Filters suitable for potable water treatment may be either the porous medium type such as diatomaceous earth filters, or non-porous medium types such as granular bed filters. The porous medium type units have specialized uses and are rarely employed, and the following section relates to the conventional and more commonly used granular bed filters. Rock filters or lake/river bottom infiltration beds are not considered equivalent to the filtration process.

All gravity and pressure filters must have piping arrangements without cross-connections; i.e. where raw untreated water may come into contact with treated water. Certain proprietary industrial filters have such piping systems, and are not acceptable for the production of potable water.

6.2 GENERAL REQUIREMENTS

Both gravity flow and pressure filters are acceptable, although pressure filters are normally used for treatment of well waters of good bacteriological quality, for hardness or iron/manganese removal, rather than for removal of higher turbidities from surface water sources.

For plant capacities below 2500 m^3/d, a minimum of two filters, each capable of independent operation and backwash, is required. Where only two filters are provided, the maximum filtration rate permitted

is 9.0 m/h and each filter should have a hydraulic capacity not less than 150% of filtration rating.

For plant capacities above 10,000 m^3/d, a minimum of three filters, each capable of independent operation and backwash, is required.

The number of filters provided will depend on the process selected; i.e. with or without prior sedimentation; the method of plant operation; the method of filter control; and the quantity of storage required.

6.3 FILTRATION RATES

The rate of filtration will depend on the quality of raw water and pre-treatment provided. The designer should also consider the effects of the type of supervision and laboratory control practised.

The maximum filtration rate allowed is 12.0 m/h. Where experimental studies using proposed source water provide sufficient data to demonstrate the acceptability of higher rates, up to 18.0 m/h is allowed.

6.4 FILTRATION SYSTEMS

When controlled rate filtration is used, it is normal to design filters to discharge to a reservoir at a lower elevation than the filter floor. For prefabricated filters, typically operated in the declining rate mode, other configurations may be acceptable.

It is recommended that filter effluent piping be designed hydraulically for flows of 50 - 100% in excess of filtration design capacity to accommodate potential peak flow demands.

Acceptable types of filter operation include declining rate filtration (which is common in package filtration plants); influent flow splitting; and constant rate filtration.

In declining rate filtration, filter influent enters through specially designed manifolds to provide equal distribution to all filters. The filter outlet contains a restriction limiting filtration rate through a clean bed to the maximum allowable rate, and filtration rate declines as the bed plugs. An advantage of this method is that it avoids potential water quality deterioration caused by high shearing forces on a dirty bed which can occur in constant rate filtration (see later) as design headloss is approached. Disadvantages include the need for special care in start-up of filters by gradual opening of the effluent valve, and a loss of operating flexibility which is particularly significant in larger plants.

Influent flow splitting employs free fall weirs as a means of providing equal flow to multiple filters, thus avoiding the need for, and cost of, effluent control systems. The disadvantages include the potential for flocculated particles to shear passing the weir, thus impairing the filtration process; and that weir settings are normally made to accommodate a single plant flow rate. Changes in plant flow cause

different losses in influent channels and the influent flow is no longer equally divided.

The most commonly used method is known as variable controlled rate filtration, or more frequently, constant rate filtration. In this method the filter effluent piping contains a flow measuring device, typically a venturi flow meter, and an automatically controlled modulating butterfly valve. The filtration rate is set to a pre-determined value by positioning the effluent valve accordingly and filtration is continued at that rate by the effluent valve gradually opening to compensate for increased head losses as the filter bed plugs. The rate is maintained until clearwell level exceeds a level typically 300 mm below the design top water level. At this point the filtration rate decreases proportionally to the 'freeboard' in the clearwell until the filter shuts down when the clearwell is full.

The control system is frequently designed to accommodate a 1 - 4 range of effluent flow.

6.5 FILTER DESIGN

The design of gravity filters should provide:

- adequate headroom above the filter to permit inspection and operation, and provide reasonable access to the filters for observation;

- protection against floor drainage entering the filter, by means of a suitable curb, or roof drainage entering the filter;

- an overflow to prevent flooding, unless provided elsewhere in the raw water supply system;

- means of cleaning influent pipes or conduits where solids loadings are high;

- effluent piping arranged to prevent backflow of air into the filter;

- means of drainage, and a waste washwater drain of sufficient capacity (see Section 6.7);

- operation with a minimum water depth in excess of the design terminal headloss to prevent negative pressure and air binding of the filter;

- an acceptable method of regulating flow as described in Section 6.4;

- indicating instruments at least for loss of head and rate of flow.

It is recognized that certain package or pre- fabricated filters do not meet all these requirements, but this does not preclude their use where deemed appropriate by the M.O.E.

Washwater troughs should be designed so that the bottom of the trough is above the level of the expanded media during backwashing.

The bottom of trough to the top level of the static media should not be less than 600 mm, and in some cases should be higher, and the trough capacity

should be such that the maximum washwater rate can be accommodated with at least 50 mm freeboard. The trough overflow level to static media top level is normally 900 - 1200 mm. The tops of troughs must be level and it is recommended that adjustable weir plates be used to ensure this. Trough spacing should be such that equal filter areas are served by each trough, and a maximum horizontal travel for suspended particles to reach a trough of 1.0 m is recommended. Alternatively, side weir designs may be used, provided sufficient operating data are available to demonstrate the effectiveness of filter washing by this method.

Sample taps should be provided, or preferably continuously running sample lines, for filtered water. Sampling at the coal/sand interface or other points of the bed is also recommended. The relative turbidity values measured permit better control of coagulant dosages. Sample lines should be kept short to minimize lag time in any control system.

In general, the foregoing applies equally for pressure filters, and in addition pressure filters should be provided with air release valves at high points, means of observing waste water during backwashing, and a suitable access for inspection or repairs. Particular care must be taken in the piping configuration to avoid cross-connections.

6.6 FILTER MEDIA

Filter media should conform to A.W.W.A. specification B100.

The selection of media size, distribution, and depths should be such that in operation the filter reaches its design terminal headloss at approximately the same time as either turbidity, colour, or iron/ manganese breakthrough - whichever is the controlling process parameter - occurs.

The media should consist of a lower level of silica sand, not less than 200 mm deep, and an upper layer of anthracite coal not less than 450 mm deep. Alternate configurations, including multi-media and proprietary mixed media designs, will be considered where sufficient data and operating experience are provided to demonstrate their suitability.

Typically, the sand should have an effective size of 0.45 - 0.55 mm, and a uniformity coefficient not greater than 1.5. The anthracite should have an effective size dependent on desired filter performance, typically 0.8 - 1.2 mm, with a uniformity coefficient not greater than 1.5. The media selection should allow for fluidization of each type equally during backwashing.

Direct filtration is more sensitive to variations in media selection, and the designer is referred to the Ministry of the Environment Research Report W54, "Operational Variables and Limitations of Direct Filtration", (6) and to the AWWA July 1980 report, "Status of Direct Filtration" (7) for additional information.

In addition to filter media, certain types of filter bottom/underdrain systems require supporting media to prevent the passage of filter media through the

filter bottom. Typically gravel support layers in three or four overlapping gradations should be provided, ranging from 2 - 40 mm size, with a total depth of 200 mm.

The most important function of the filter bottom or underdrain is to provide uniform distribution of backwash water and/or scouring air. Proprietary filter bottoms of perforated clay or concrete are acceptable for water only wash, as are nozzles or strainer type devices installed above a plenum chamber. Where strainers are used, there may not be a need for supporting gravel layers in the filter.

Porous plate bottoms should not be used with waters high in iron or manganese, lime softened waters, or those susceptible to algae growths.

The filter bottom design should be such that essentially all head losses on backwashing occur at the final openings to ensure an even distribution of washwater.

6.7 FILTER WASHING

Provision should be made for upflow washing of filters at a rate which will expand the filter media 25 - 40% overall, typically 30%. The wash rate required to provide this expansion will vary with the depth and type of media. In general, a rate of approximately 45 m/h will be adequate with a 10°C washwater temperature, and equipment capable of washing filters at the higher rates required as washwater temperature increases should be provided.

Filtered washwater may be provided at the required rates either from a washwater tank, or preferably by washwater pump.

Where washwater pumping is proposed, a duplicate pump is preferred unless an alternate source of sufficient quantity is provided by other means.

The use of a high pressure source, even with pressure reducing valve, is not permitted for gravity filters.

Where washwater supply tanks are proposed, the tanks should be elevated at least 8 m above the washwater troughs. The washwater storage volume should be 15 m3/m2 filter area except where air scour systems are used, in which case the quantity of washwater may be reduced. The size of washwater tank will depend on the number of filters to be washed, the sequencing and frequency of washing, and the recharge rate for the supply tank.

It should be noted that a number of commonly used proprietary filters with integral washwater storage exist. In such cases, the design terminal filter headloss is frequently less than 1.5 m, and lesser quantities of washwater are provided. Such systems are acceptable where sufficient operating data are provided to demonstrate the long-term effectiveness of filter washing.

In all cases some means of flow regulation must be provided to obtain the desired rate of filter wash, and the operation or control of the system should be such that the washwater flow rate can be gradually increased at the start of a wash to prevent upsets of the bed, and gradually decreased at the end of the wash to allow the bed to stratify properly. The washwater line should be equipped with provision for measuring and indicating the rate of flow with reasonable accuracy.

The design of the washwater supply piping should ensure that no unwarranted slugs of air are introduced into the bed, since considerable disruption of the media will result. Typically, entrapped air problems may arise when vertical turbine style washwater pumps are used, or where dissolved gases are released on standing and warming of water held in the washwater supply piping.

Surface or auxiliary wash systems are required for all gravity filters without air scour. While such washing may be accomplished by a suitable hose system, it is preferred that a system of fixed or revolving nozzles be provided to wash the filter surface. For some types of treatment processes it

is possible for particular matter to penetrate deeply into the filter bed and additional subsurface washing may be needed. The systems should be designed for a minimum 600 kPa water pressure, and have approved devices to prevent backsiphonage.

Alternatively, where the filter bottoms are suitably designed, a combination air and water wash may be provided. Such systems are commonly used where reduction of waste washwater quantities is a consideration, or deep penetration of "sticky" material into the bed could be expected. Typically, a 20 - 30% reduction in wash volume may be achieved with the use of air scour systems.

The control system for combination air and water wash should be designed to allow air and water to be applied to the filter either singly or in combination.

The air supply system should be capable of providing 50 - 60 m^3/h air per square metre of surface area, and be fully adjustable. The maximum recommended supply pressure is 100 kPa. If compressors are used for this supply, they must be of the oil-less type.

SECTION 7

7.0 TREATED WATER SYSTEMS

7.1 TREATED WATER PUMPING

In general, the requirements for a treated water pumping station are similar to those outlined in Section 4.4 for raw water pumping stations.

The minimum number of pumps to be provided is two, in addition to any pumps required to provide fire flows. The minimum capacity shall be equal to the maximum day demand, and the actual capacity will be dictated by the distribution system and storage design and capacities. In each case this capacity shall be a firm capacity, i.e. with the largest unit out of service.

Generally, pumps should be selected which have maximum efficiencies at the average head condition, but which can meet the maximum flow and pressure conditions. Particular attention should be paid to pump specifications with respect to available net positive suction head.

Pumping station headers should be adequately protected from transient pressure surges which may occur if pumps stop instantaneously (on power failure). Protection may be provided either by appropriate valves or hydraulic transient surge tanks. Any discharge from such a system may be connected directly back to the treated water well or storage reservoir, or may discharge to a drainage system provided that an adequate air gap is included to prevent backflow.

The designer should carefully review potential operating levels of pump wells or storage reservoirs, and pump elevations, to ensure that an adequate priming system of suitable capacity is provided.

Pump discharge valves should be slow acting type, properly controlled to avoid high transient pressures in the system on opening or closing. These valves should be interlocked with the pump operation to provide such protection.

The design of the treated water well should conform in all respects to the recommendations of the Hydraulic Institute. Where backwash pumps draw from the same well as treated water pumps, the variations in well level, and consequent variations in suction requirements must be considered when selecting the backwash pumps.

7.2 IN-PLANT STORAGE

Treated water storage will normally comprise the clearwell, treated water pump well and storage reservoir. The volume of storage required must be such that distribution system needs and in-plant needs, such as filter washing, can be met from storage without the need for filters to operate at fluctuating rates to meet such peak demands. The volume considered in this case is the effective storage volume; i.e. the volume between design maximum level and the minimum level to which the storage components may be pumped down. The storage volume must also be adequate to provide at least the retention time required for chlorination, even at low operating levels. In

this case the total storage volume may be considered. (See Appendix D). Typically, total storage of 10 - 20% of treatment capacity is sufficient to meet these criteria. It should be noted, however, that system needs in excess of maximum day requirements are normally met from elevated storage since this is inherently more secure in terms of the protection offered.

The designer should consider providing sufficient flexibility, by virtue of piping arrangement or multiple cells, to permit the plant to continue to operate for short periods with either the clearwell or storage reservoir out of service. The design should include separate inlet and outlet pipes or conduits arranged to promote circulation within the reservoir, and to avoid dead spots.

The design objective for water storage components is to minimize the chance of contamination of the finished water. The designer should refer to the Ministry "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems, and Storm Sewers" (3) prepared in May 1979 for specific requirements for water storage facilities.

7.3 PRESSURE STORAGE TANKS

Typically, pressure storage tanks are used in small closed systems to maintain acceptable system pressures without the need for continuous pumping.

Total storage requirement should be based on meeting maximum demand excluding fire flow requirements which are typically provided by by-passing the storage tanks. The volume required will depend on the system design pressure low and high limits, and a recommended minimum pump cycle time of 10 minutes, unless special motors are specified.

Typically, total storage of 1 m^3 is required for each 150 m^3/d capacity of the lead pump, and 1 m^3 usable storage required for each 600 m^3/d capacity of the lead pump.

Tanks should be provided with a pressure gauge, automatic or manual air blow-off, and a drain. Mechanical means of adding air to the tanks should be provided. By-pass piping should be included to allow the system to operate with a tank out of service.

Tanks must be installed within buildings and comply with all regulations governing the construction and installation of pressure vessels.

SECTION 8

8.0 CHEMICAL SYSTEMS

8.1 GENERAL

Normally, only those chemicals for which AWWA standards exist should be used. Under special circumstances, other chemicals may be used for which standards have not been developed, provided that adequate toxicological data are presented which demonstrate that the proposed chemical, the applied dose, and the method of manufacture will result in a finished water which is acceptable from a public health standpoint.

The general types of chemicals required will depend on the process requirements, and the specific chemicals selected for treatment will depend largely on safety and economic considerations; for example, chlorine disinfection may be accomplished by either liquid sodium hypochlorite, solid calcium hypochlorite, or liquified chlorine gas supplied in various container sizes.

Design information is readily available from chemical suppliers, and additional information may be obtained from standard tests such as:

"Dangerous Properties of Industrial Materials", (9) N. Irving Sax.

"Hazardous Chemicals Data 1975" (10) - U.S. National Fire Protection Association, booklet #49.

The designer should ensure that the chemical systems and auxiliary equipment complies with Bill 70 - The

Occupational Health and Safety Act, 1978; and Regulations for Industrial Establishments.

8.2 CHEMICAL STORAGE

Storage is usually provided for thirty days' consumption at the maximum anticipated chemical usage rate, allowing for variations in chemical dosage and flow in that period. Where deliveries of chemicals can be expected to be interrupted by adverse weather conditions in isolated locations, provision should be made for increased storage capacity. Where deliveries at short notice can be assured, and the material is not essential to the production of safe water, storage requirements may be reduced.

Except where economically impractical, sufficient storage should be provided to permit full load deliveries. Typically, these will be 20 tonnes for common dry chemicals and 15 000 - 25 000 L for liquid chemicals. The minimum recommended storage for truckload delivery is 25% greater than one truck load, or one truckload plus the quantity of chemical consumed in seven days, whichever is greater.

Storage of chemicals inside buildings is recommended to avoid problems of materials freezing or becoming too viscous to pump, vandalism, high costs of providing fully weatherproof equipment, and the difficulty of containing chemical spills, particularly gaseous spills, which may occur.

It is recommended that the chemical storage area be segregated from the main areas of the treatment plant, and that separate storage areas be provided

for each chemical. Where chemicals in storage could react dangerously with other materials in storage, e.g. chlorine and ammonia, strong acids and alkalis, segregated storage is required. The storage and feed equipment areas should be arranged for convenience of operation and observation, and located so as to provide easy access for chemical deliveries.

It is strongly recommended that all chemical storage be at or above the surrounding grade. Where subsurface locations for chemical storage tanks are proposed, these locations shall be free from sources of possible contamination, and assure positive drainage for ground waters, chemical spills, and overflows. Where above grade storage is provided, due consideration should be given to the method of unloading chemicals; for example, there is a limit on the allowable pressures to be used for air-padded trucks. Where drums or dry bagged chemicals are used, it is recommended that some form of loading dock or ramp be provided.

Storage areas should be arranged to prevent any chemical spills, or results from clean-up operations, entering the water under treatment. Floor surfaces should be smooth and impervious, slip-proof, and sloped so as to drain rapidly.

Chemical ventilation systems should be arranged so that air is exhausted outside the building and slight negative pressures are maintained where dry chemicals are in use, as a dust control measure. Where large amounts of dust are anticipated, appropriate local exhaust systems and filters or scrubbers should be provided in the ventilation system. Ventilation

systems should be designed specifically for corrosive service and special measures taken in dust systems to prevent static build-up or other explosive potential.

The designer should note that special precautions may be necessary in the design of both normal and emergency ventilation systems to prevent chemical concentrations in the atmosphere from exceeding limits permitted under the Environmental Protection Act, or which might be hazardous.

A common problem arises with ventilation of chlorine gas storage areas where the requirements of the Ministry of Labour are for thirty air changes per hour in emergencies, and the potential exists for environmental damage to be caused by release of large quantities of chlorine gas into populated areas. Such problems should be reviewed on a case-by-case basis, since the scrubbing equipment necessary can be very expensive almost regardless of plant size.

Chemical buildings or storage areas must be provided with eye-wash and/or deluge showers, adequate facilities for cleaning up chemical spills, space for cleaning and storage of the recommended protective equipment, and adequate warning signs, conspicuously displayed where identifiable hazards exist. It is recommended that all doors in chemical buildings open outward, and that corridors or space between storage areas be a minimum 1.5 metres wide to permit the use of hand trucks, etc. for safe movement of materials.

8.3 LIQUID CHEMICALS

All storage tanks should be provided with an adequately sized fill line, minimum 50 mm diameter, sloped to drain into the tank. The fill line should be adequately identified at the end remote from the tank, and provision should be made to drain this fill line, if a "down leg" exists.

Each tank should have an adequate vent line, minimum size 50 mm, with a down-turned end. Where venting outside the room is required, the vent should be provided with an insect screen.

All tanks should have an overflow adequate for the rate of fill proposed for the tank, sloped down from the tank, with a down-turned end and free discharge, located where it can be readily noticed. Overflow pipes will not be permitted to connect directly to the sewer, and where they pass into a receiving sump or conduit, they must terminate at least two pipe diameters above the maximum level in the sump.

Each tank should be provided with means to indicate the level of contents in the tank, and where an external level gauge is provided, a shut-off valve at the tank connection is recommended. Each tank should be provided with a drain, which is not permitted to discharge directly to a sewer, and must terminate at least two pipe diameters above the overflow rim of a receiving sump. Tanks should be provided with removable lids or covers with manholes where the contents are such that venting indoors is permitted. In the case of tanks which are to be vented outside, the covers or manholes should be

constructed so as to be air tight. Tanks should be provided with an overflow not less than 300 mm above the design level when the tank is filled by pumping, and not less than 150 mm when the tank is filled by gravity.

Where lined tanks are proposed, it is recommended that weep holes in the outer shell be provided to give an indication of liner leakage.

All storage tanks should be conspicuously labelled with the name of contents and principal potential hazards indicated.

8.4 DRY CHEMICALS

Where dry chemicals are to be used, provision should be made to minimize handling and dust problems. Granular materials are preferred to powders.

Particular care should be taken to protect mechanical and electrical equipment from fine dust. Where exhaust fans, filters, and conveying systems are used, grounding must be provided to prevent the build-up of static electricity.

Bulk storage silos should be provided with adequate sized fill openings, and fill lines where necessary should be smooth internally with long radius elbows.

Silos should be provided with suitable level indicating devices, such as load cells. They should include a pressure relief valve when pneumatic fill systems are provided.

The designer should take note of the material characteristics such as flowability, tendency to pack tightly, angle of repose, etc. in the design of the silo bottom and method of removal of material to a feeder. Provision should be made to relieve bridging or rat-holing of the stored material, either by manual, mechanical, or other means of agitating the hopper bottom or improving flowability of the material, for example by air fluidization.

Silo vent and exhaust systems should be provided with dust filters and/or cyclone type separators to prevent the release of dust into the atmosphere.

8.5 GASEOUS CHEMICALS

Chlorine, and to a lesser extent, sulphur dioxide, are the most commonly used gaseous chemicals; ammonia and carbon dioxide gas systems may be required by some treatment processes. The designer should obtain design information for these system from the suppliers.

Gas storage areas should be separated from the other areas, with separate outside accesses, and arranged to prevent the uncontrolled release of spilled gas to other areas of the plant.

Means of measuring the contents of gas containers should be provided, and where necessary for the proper operation of the feed system, means of adjusting and indicating gas pressures should be provided. A system for automatic changeover when gas cylinders are empty is recommended.

Where high feed rates are required, it may not be possible for thermodynamic reasons to withdraw the required gas quantity from a single cylinder. The designer should consider either using multiple cylinders on-line or the use of an evaporator to meet higher withdrawal rates.

The designer should allow sufficient space in the storage area for convenient moving of cylinders from full storage to on-line to empty storage.

Special requirements exist for chlorine systems, and the designer is referred to the Ministry of Labour Engineering Data Sheet 5-3, "Storage and Use of Chlorine", (8) and the Ministry of the Environment Technical Bulletin 65-W-4, "Chlorination of Potable Water Supplies" (Appendix "D") for further information.

8.6 CHEMICAL FEED EQUIPMENT

The design and capacity of feed equipment should be such that it can supply the required quantity of chemicals at a continuous and accurate rate at all times. Equipment must be capable of proportioning the chemical feed rate to flow, and adjustment for dosage through the anticipated range.

Where the chemical added is necessary for essential processes such as disinfection or coagulation, a minimum of two feeders should be installed for each application; i.e. three chlorinators where both pre- and post-chlorination are practised. Standby units must have sufficient capacity to replace in-service units.

Feeders may be either manually or automatically controlled and capable of manual override. The control system should be such that manual restarting is required after a feeder shut-down. Coagulant feed by on-off control such as time duration signal is not allowed. Feed systems should be equipped with means to measure their output volume or mass on a test basis.

Positive displacement reciprocating type feed pumps should be used for liquid chemicals, but will not be approved for chemical slurries for which positive displacement rotary pumps should be used. Volumetric or gravimetric feeders are suitable for dry chemicals and should provide effective means of dissolving or dispersing the material prior to addition to the water under treatment. The use of remote ejectors and transmission under vacuum is recommended for gaseous chemicals to avoid pressure lines passing through the plant.

Where solution tanks are in use, means should be provided to maintain a uniform strength solution, and continuous agitation should be provided to maintain slurries in suspension. Normally, two solution tanks will be required to ensure continuity of supply when servicing one solution tank. Each tank should be provided with a drain. Make-up water for the solution tank should enter the pipe above the maximum solution level not less than 150 mm above, or two pipe diameters, whichever is greater, unless the make-up water supply has an approved backflow preventor.

Where the design of the chemical feed system includes day tanks, such day tanks should have a capacity equivalent to the maximum quantity of chemical consumed over a 30-hour period. Day tanks should either be scale mounted or have a calibrated level gauge provided. The piping arrangement for refilling the day tanks should be such that it will prevent over-filling of the tanks. In all other respects the requirements for day tanks should conform with the requirements for bulk storage tanks.

All positive displacement pumps should be equipped with adequately sized pressure relief valves. If the pumped fluid is relieved through this valve, it must pass to a safe location, preferably back to the storage tank. Where liquid filled diaphragm pumps are in use, the over-pressure should be relieved by discharge of the motive fluid to a safe location. Where oil-filled diaphragm pumps are used, the oil must be of a grade suitable for use in potable water supplies (USFDA approved). Pressure relief valves should be set not greater than 10 per cent higher than the pump discharge pressure in normal operation. Double diaphragm pumps are recommended for corrosive chemical pumping.

Where reciprocating type pumps are in use, it is recommended that flexible connections be provided on the pump suction and discharge, to prevent the transmission of vibrations to the feed line. These flexible sections should be sufficiently rigid to withstand both the pump suction and discharge pressures, and reinforced hose is recommended. The pump, in combination with its suction piping and valving arrangement, should be such that the pump

discharge rate is not affected by fluctuations in storage tank level.

Chemical feed lines should be kept as short as practical, especially suction lines, protected from freezing, and located to be readily accessible. The minimum line size should be 12 mm, unless the material in transport has a tendency to form scale, in which case a minimum size of 25 mm is recommended.

The designer should allow for line flushing, particularly for lime feed systems, or where hard water supplies are used in solution preparation which could promote scaling.

Where feed lines are provided from multiple feeders or distributed to multiple application points, adequate valving must be provided to isolate appropriate sections of the supply system.

8.7 CHEMICAL APPLICATION POINTS

Chemicals should be applied at such points, and in a manner, which ensures the maximum efficiency for treatment and maximum safety for both operators and consumers.

Except where presedimentation is practised, applying chemicals prior to screening is not recommended.

Potentially corrosive chemicals should not be applied immediately preceding screens or pumping equipment, nor should solids producing materials be applied prior to pumping.

Application points should be selected for high turbulence locations such as at pipe constrictions or hydraulic jumps, or through a suitably designed diffuser. Application near points where water flow splits should be avoided whenever possible.

Alternative application points for each chemical should be provided for greater operating flexibility.

The sequence of addition of chemicals is often very important. For example, the use of activated silica and alum for treatment of low turbidity waters will have (different) successful results if the activated silica is added before or after the alum, but would be ineffective if added at the same time as alum since they react together.

Backflow or siphonage between multiple feed points should be avoided, and where chemicals are added to pressurized lines, isolating valves should be provided.

The addition point for chlorination should be selected to ensure adequate disinfection and reduce the potential for formation of chlorinated organic compounds.

Where fluoride chemicals are used, it is recommended that they be added in a way to preclude any possibility of siphonage, typically by injection into a pressurized line or by using an air-break.

SECTION 9

9.0 WASTE HANDLING & DISPOSAL

9.1 GENERAL

As a general requirement, all wastes produced from a water treatment plant will require treatment. The degree of treatment necessary will be dictated by the requirements of the watershed, and this should be established through consultation with the appropriate Ministry Regional office.

It is assumed that the process and plant design, and the intended operation are consistent with minimizing the quantity of waste to be treated. For information on such reduction, the designer may refer to the AWWA 1973 seminar proceedings, "Minimizing and Recycling Water Plant Sludges" (11); the AWWA 1978 seminar proceedings, "Water Treatment Waste Disposal" (12); and the Canada-Ontario Agreement Report No. 77, "Treatment and Disposal of Water Plant Wastes in Ontario" (13).

Frequently, the waste treatment method will be economically dependent on the available options for final disposal of the residues remaining after treatment of the plant wastes. The designer's options may be severely limited by the economically viable disposal alternatives for his particular case. The designer should ensure that the overall design of the waste treatment process is cognizant of these alternatives.

9.2 TREATMENT OPTIONS - MINOR WASTES

All sanitary wastes from water treatment plants must receive treatment either by discharge directly to a sanitary sewer system, or to an approved individual waste treatment facility providing suitable treatment.

Sanitary wastes should be kept separate from all process wastes to avoid the need for treatment of all plant wastes in the same manner as sanitary wastes.

9.3 TREATMENT OPTIONS - MAJOR LIQUID WASTES

Filter backwash water may discharge directly to a sanitary sewer, or that part of the washwater waste which exceeds criteria for discharge to the raw water source may discharge directly to the sewer. Since washwater discharge rate is frequently in the same order of magnitude as the plant capacity, these courses of action are only available where connections can be made to major sewerage systems which can withstand the hydraulic surges imposed on both the sewers and sewage treatment plant.

More commonly, backwash waste is discharged to a waste holding tank for further treatment, or to a waste holding lagoon for settling (as discussed later). The size of the waste holding tanks will depend on which processes are to occur in the tank, or which processes follow later in the waste treatment scheme; the rate of withdrawal of material from the tank; the interval required between filter

wastes. Typically, the tank volume is equal to the wastewater produced from washing two filters.

The washwater tank enables subsequent treatment stages to operate at continuous low rates much lower than filter backwash rates; i.e., surges are removed, and efficiency improved. Influent arrangements to holding tanks should be designed to promote bottom scour to prevent solids build-up in the tank.

Waste in the holding tank may be permitted to settle and supernatant discharged to the source water or sanitary sewer, and the settled sludge pumped out for further treatment. Alternatively, the contents of the waste holding tank may be pumped out to a wastewater thickener, typically a picket-fence type clarifier, to produce an overflow which can be discharged to the source water or sanitary sewer, and an underflow which is concentrated enough to be combined with settled sludges from other plant processes; i.e. clarifier blowdown or sedimentation tank sludges. Typically, these sludges have a concentration of 1 - 2% and are ready for further treatment.

9.4 <u>TREATMENT OPTIONS - SLUDGES</u>

Where large quantities of sludges result from the treatment process, i.e. in excess of one tonne/day, the designer should consider the possibility of coagulant recovery. Acidification of alum sludges is a demonstrated process which can lead to substantial recovery of coagulant which can be recycled. The recently developed liquid ion-exchange process may also be economically viable for alum recovery.

Where coagulant recovery is proposed, the effect of recovery on other sludge constituents, e.g. iron, organic colour materials, should be considered in view of maintaining treated water quality when using recovered coagulants.

Other alternative treatments for sludges are designed to reduce the volume of material, and hence the cost, for final disposal. For large plants mechanical dewatering methods such as centrifuges or filter presses may be appropriate. For medium and small plants the natural freeze-thaw method (see Section 9.8) may be more economical.

9.5 TREATMENT OPTIONS - FINAL RESIDUES

At the conclusion of treatment of plant process wastes, a solid, semi-solid, or semi-liquid material will remain for final disposal.

Subject to operating conditions at nearby sewage treatment plants, semi-liquid sludges may be added to the sewage plant at an appropriate stage of that plant's treatment. The designer should carefully review the operation of the sewage plant, and the method of sludge disposal from that plant prior to selecting this disposal option.

Alternatively, solid and some semi-solid sludges may be acceptable at sanitary landfill sites, or approved chemical landfill sites.

9.6 OTHER TREATMENT OPTIONS

Under certain conditions, it may be possible to recycle part or all of the backwash waste or sludges into the water treatment process. In some cases the treatment efficiency can be improved by addition of these wastes to provide nucleation sites for flocculation, or to provide more rapid settling.

Provided that the designer can demonstrate that recycling will not adversely affect either water quality, plant operations, or production, such applications may be acceptable.

9.7 WASTE PIPING DESIGN

The designer should consider the provision of a filter backwash waste by-pass to source to allow treatment plant operations to continue at all times, even if the waste treatment system is not operating.

Piping for wastes is not allowed to pass through finished water retaining structures. The protection of finished water quality should be ensured by the design proposed.

Waste piping should be provided with adequate clean-outs and provision for flushing. A minimum velocity of 0.5 m/s is recommended for all wastewater lines.

Where waste outfalls are permitted, they should be located so that discharge of wastes does not adversely affect the quality of raw water provided to the

plant, and appropriate measures should be taken to ensure proper diffusion of the waste into the receiving water.

9.8 LAGOON DESIGNS

Where waste treatment through lagoons is proposed, a minimum of two cells should be provided, each capable of independent operation. Each cell should be sufficiently large to hold twelve months' sludge production plus a minimum of one day liquid waste volume.

Sludge production typically may be calculated as 0.4 times the dry weight of alum added, plus 1.0 times the turbidity. In storage, the sludge concentration will typically average 3 - 4% (from 0.5% for "fresh" sludge and 7 - 8% for old sludge). The designer should confirm the actual sludge production from plant or pilot tests.

Typically, the lagoon design will allow sludge depths of 0.5 - 0.75 m, supernatant depths of 0.5 - 0.75 m and allow for ice cover as appropriate to local conditions.

Inlet piping should be designed to distribute the incoming waste uniformly and minimize disruption of the settled sludges. The piping should be designed to be free draining to reduce the possibility of frost or ice damage in winter.

Outlet piping should be designed to permit displacement operation during winter, and should be free draining. Each cell should have a supernatant decant system which is adjustable.

SECTION 10

10. PIPING

10.1 GENERAL DESIGN

All piping used in water treatment plants should be manufactured in accordance with AWWA, CSA, CGSB, ASTM or other internationally recognized standards. Material selection will depend upon economic and corrosion factors, as well as the type of equipment used and connections required. The designer should be aware of the greater potential for deflection in thin wall pipe systems than in other systems.

In the design of the piping, due allowance should be made for future capacities and also the ease of extending this piping without major disturbance to the plant. In the general piping arrangement, sufficient space should be provided for piping to be removed, and the pipe design should provide for the proper isolation, through valves, of pipe sections to enable them to be repaired or replaced. The designer should allow for the possibility that piping could be installed during construction when temperature conditions could be substantially different from the design condition (for example, piping could be installed in temperatures anywhere between +40°C and -20°C), and substantial differences in pipe dimensions could occur. For this reason the use of PVC pipe with cast iron mechanical joint fittings should be avoided. Where piping is cast-in-place, due allowance should be made for differential expansion between pipe material and structures.

Piping should be arranged so that all valves, flow meters, and other items which may require regular

inspection or maintenance are conveniently accessible. Piping should be provided with drains at all low points, and air release valves at all high points. Sludge piping should be provided with cleanouts and flushing facilities.

The design of the piping should allow for proper restraint under all anticipated conditions, particularly where surges may occur and high transient pressures could result, or where different temperatures occur seasonally.

Where piping connections are made between adjacent structures, at least one flexible coupling should be provided if any possibility of settlement exists. Particular attention should be given to pipe bedding in areas adjacent to structures to avoid damage due to settlement.

10.2 SIZING

Piping should normally be designed for flows as listed below, with a minimum size of 100 mm:

Table 10.1
Piping design for flows limited by Process Considerations:

Item	Maximum Velocity: m/s
Flocculated Water	0.60
Pre-settled water	0.60
Post-settled water	0.30
Filter influent	0.20

Table 10.2

Piping design for flows limited by Hydraulic Considerations:

Item	Maximum Velocity: m/s
Raw water (pumped)	3.0
Filter effluent	2.0
Wash supply	3.0
Wash drains	2.5
Treated water (pumped)	3.0

Table 10.3

Piping Identification Requirements

Contents Classification	Lettering Colour	Background Colour & C.G.S.B. Equivalent	
Dangerous materials	Black	Yellow	505-101
Safe materials	Black	Green	503-107
Protective materials	White	Blue	202-101
Fire protection	White	Red	509-102

Piping may be identified by complete painting of the line or, in cases where painting is not required, vinyl impregnated adhesive cloth tape is recommended. Where there is no previously existing standard colour code, it is suggested that the following standards apply. Where alternative chemicals are used, it is suggested that a coherent code be developed which consistently identifies the degree of hazard associated with the material contained.

Table 10.4
Process Piping Colour Code

Contents	Colour
Raw Water	Dark Blue
Settled Water	Mid Blue
Finished Water	Light Blue
Backwash Waste	Mid Brown
Settled Backwash	Light Brown
Sludge	Dark Brown
Drainage	Light Grey
Sanitary Waste	Black

10.3 SANITARY PROTECTION

Within a water treatment plant considerable potential exists for cross-connections between potable and non-potable waters. Typical examples are potable water supplies for chemical solution make-up; cooling water supplies to mechanical equipment; seal water supplies to pumps; and filter surface wash piping. While pump seal water supplies need only be of better sanitary quality than the water pumped, it is frequently more convenient to use the finished water system to provide seal water.

The designer should consider avoiding cross-connections and protecting plant staff by providing two separate in-plant water supplies from the high pressure system. One major supply with a reduced pressure type backflow preventer of approved design for "process uses" such as solution preparation, cooling, etc., and a second supply with a backflow preventer for "potable uses" such as drinking and

sanitation. The two supplies should be clearly distinguished as potable and non-potable, since internal cross-connections within the process water may occur.

10.4 PIPE CODING

Piping identification is mandatory as required by the Occupational Health and Safety Act. It is recommended that all process piping be adequately identified as to contents and direction of flow to simplify operating and maintenance procedures, and improve safety.

To comply with CSA B53, clearly visible lettering should be used to indicate contents as outlined below:

Table 10.5
CHEMICAL PIPING IDENTIFICATION

Material	Primary Colour	Secondary Colour
Aluminum Sulphate	Light Green	-
Ferric Chloride	Light Green	Orange
Silicate Compounds	Light Green	White
Polyelectrolytes	Light Green	Grey
Chlorine Gas*	Yellow	-
Sodium (Calcium) Hypochlorite	Yellow	White
Solution	Yellow	Red
Chlorine Dioxide Solution	Yellow	Orange
Lime	White	Orange
Sodium Carbonate	White	Grey

Sodium Bicarbonate	White	Yellow
Carbon Dioxide*	White	-
Alkali Hydroxide	White	Red
Sulphuric Acid	Orange	Red
Sulphur Dioxide*	Orange	-
Ozone*	Brown	-
Potassium Permanganate	Purple	-
Fluoride Chemicals	Purple	Red
Ammonia*	Bright Blue	-
Flammable Gas	Red	-

*ALL GAS SOLUTION LINES TO HAVE LIGHT BLUE SECONDARY

Table 10.6

STANDARD PIPING COLOURS

Colour	C.G.S.B. Standards
Grey	501-103
Light Grey	501-108
Dark Blue	502-103
Bright Blue	502-104
Light Blue	502-106
Mid Blue	502-208
Light Green	503-323
Dark Brown	504-102
Brown	504-105
Mid Brown	504-107
Yellow	505-101
Light Brown	505-206
Orange	508-102
Red	509-102
Purple	511-101
Black	512-101
White	513-101

SECTION 11

11 GROUNDWATER SYSTEMS

11.1 GENERAL

While other sections of these guidelines apply equally to surface water sources and groundwater sources, there are a number of special considerations which relate to groundwater systems and which should be reviewed by the designer.

Acceptable groundwater supply structures may consist of conventional vertical wells, horizontal collector wells, infiltration galleries or spring encasements.

The designer is referred to the joint Municipal Engineers Association/M.O.E. publication "Recommended Guidelines for Small Groundwater Supply Systems for Residential Development", (14) August 1981 revision, for considerable detail on well system design.

11.2 PRE-DESIGN STUDY

Prior to the design of a supply structure, adequate geological, hydrological, and water quality data on the aquifer should be obtained to allow review and approval of the proposed groundwater source.

It is recommended that these data be obtained by or under the direction of a qualified hydrogeologist. The specific water quality sampling programme and analytical requirements should be prepared after consultation with the local district office of the Ministry of the Environment.

In particular, the studies should address such factors as aquifer hydraulics and safe well yields, potential sources of contamination, and water treatment requirements.

11.3 WELL DESIGN

The design objective for the well must be to provide a hydraulically efficient and structurally sound well that will produce the required water quantity on a continuous basis, and which is safe from external contamination.

For specific details of design and construction criteria which will produce a finished supply structure which complies with these requirements, the designer should review:

AWWA A-100, Standard for water wells

O.W.R. Act, Section 40, 1970

Ontario Regulation 648/70.

11.4 WELL PUMPHOUSE DESIGN

The use of well pits to house pumping equipment is discouraged because of the sanitary, maintenance, and safety problems associated with this type of construction.

In general, the design criteria for well pumping stations follow those presented for raw and treated water pumping stations presented earlier. In

addition, the following special considerations apply to wells.

The elevation of the top of the inner well casing should be above the existing ground level, the normal flood level of and adjacent surface water body and at least 0.15 metres above the floor to prevent flooding of the structure.

A pump pedestal should be provided around the casing to support the full weight of the pump and to prevent any weight from being placed on the working casing or any associated well casing.

Where wells are completed in flowing artesian conditions, piezometric control of the aquifer is required. This may be achieved by installing a suitably sized, valved discharge-to-waste line to convey water from the inner well casing to outside the building.

A water-tight seal should be provided between the pump base plate or submersible discharge head and the pump pedestal or between the well casing and the pump discharge column to prevent the entrance of contaminants.

An aperture for air venting must be provided to the inner well casing. Where there are indications of excessive quantities of explosive or toxic gases in the water both the inner and outer well casings must be vented to the outside of the pumphouse.

Cross-connections or return pipes that will permit water to be recirculated back down the well should be avoided as they may cause damage to the well.

Typically, the well should be located within 1.2 metres of an exterior wall of the pumphouse and centered under a hatchway in the roof, at least one metre square, to facilitate access by crane.

The piping layout in the pumphouse should include an in-line free discharge pipe to the outside of the building to permit future testing of the well. The end of the pipe should be equipped with a free discharge pipe orifice and manometer tap, calibrated to the design yield of the well. If high static water levels exist, the designer should consider the use of a by-pass to waste from the pump to avoid transient high discharge pressures on start-up.

A combination flow controller, with pressure gauges upstream and downstream, a suitable check valve, and an indicating flowmeter should be installed in advance of the free discharge pipe.

Well water quality monitoring should be provided by including a suitable sampling point. Water level monitoring should be provided by including at least one opening in the well head, typically 25 mm diameter, which allows vertical access to the inner casing for equipment installation. Additionally a water level measuring airline should be installed, clamped to the pump column, complete with a suitably calibrated pressure gauge.

11.5 TREATMENT

The minimum treatment for groundwater is disinfection. Where necessary, additional treatment should be provided to ensure that the finished water meets the quality guidelines described earlier. Such additional treatment will normally be directed at reductions in dissolved gas, iron and/or manganese reduction. Where possible, the selection of the treatment process should be based on experimental or comparative data, and the process selected from alternatives should be that which minimizes overall costs.

Where aeration is proposed for oxidation, gas stripping, or taste-odour control, forced draft or induced draft systems are recommended. Consideration will be given to approval of other aeration methods. The designer should note that air exhausts from aeration systems may be considered as plant emissions requiring compliance with the quality limits established by the Environmental Protection Act.

Acceptable methods for reduction of iron or manganese concentrations include oxidation by aeration, chlorine, permanganate, and ion exchange. Control by sequestration is also acceptable.

Where the treatment process results in a precipitate, filtration should be included to provide protection for the water distribution system.

SECTION 12

12.0 STANDBY POWER

The need for standby power and the extent of equipment requiring operation by standby power must be individually assessed for each water treatment plant. Some of the factors which will require consideration in making the decisions regarding standby power and the process units to be operated by the standby power equipment are as follows:

- frequency and length of power outages in the area served by the water treatment plant;
- reliability of primary power source;
- number of power feeder lines supplying grid system, number of alternate routes within the grid system, and number of alternate transformers through which power could be directed to the water treatment plant;
- available finished water storage within the system; type of water storage (underground or elevated);
- requirements for fire protection;
- type of standby power.

It is desirable to provide finished water storage in a municipality in accordance with the Ministry's "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems, and Storm Sewers". (3) If such storage capacity is provided in the system (elevated or

ground storage with standby power), standby power is generally not required to ensure uninterrupted operation of any treatment units and pumping equipment.

In direct pressure systems, a diesel-driven fire pump at the treatment plant is provided for a dual purpose: (a) pumping fire flow; (b) providing emergency service during power outages.

Depending on the complexity of the plant, standby power may or may not be provided for auxiliary services such as lighting, instrumentation, and control. In some cases, emergency lighting for safety purposes by means of a battery pack may suffice.

It should be noted that it is standard practice when running diesel engines to permit them to run for not less than 30 minutes under at least 50% load to ensure self-cleaning. For this reason the designer should give consideration to providing a standby power system which requires an operator decision to start up in manned plants, and a timer system in unmanned plants. Timers should also be provided to bring equipment on line in such a way that the generators will not be overloaded by the starting current requirements of motors. Similar protection will be necessary to avoid overload of the normal electrical supply on resumption of power following a power failure.

For the suggested design of diesel-powered standby power equipment, reference should be made to the Ministry's "Standard Specification for Diesel Engine Generator Sets". (15)

The standby power equipment should preferably be located so that it fits conveniently into the electrical distribution system of the plant. Under some circumstances, it may be preferable to locate the diesel generator set remote from the treatment plant in a separate building in order to satisfy MOE air requirements.

Engine cooling water may not be returned to any process units. Where cooling water is passed to a sewer system, the designer should ensure that the sewer system has adequate storage or standby power, or an alternate diversion of cooling water will be required.

Diesel units should be mounted on a pad and surrounded by a retaining curb which can retain any fuel spills likely to occur. The unit should be located so as to provide a clear space for inspection and servicing not less than one metre all round the unit.

Sufficient fuel storage should be provided, taking into account the historical data on length of power outages in the area, and any weather or other conditions which might preclude fresh deliveries of fuel. A minimum 500 L storage should be provided for generator set capacities up to 25 kW and 1000 L storage for set capacities from 30 to 100 kW.

Either underground or inside fuel storage tanks may be used. In considering which type to use, factors such as corrosion potential, consequences of leakage, storage volume needed, need for fuel pumps, etc.

should be evaluated. For details on the requirements for underground storage tanks refer to ULC - S603.1 - 1977 "Standard For Protected Steel Underground Tanks for Flammable and Combustible Liquids", (16) Underwriter's Laboratory, Canada.

The location of the standby power system should be such that site perimeter noise levels will be in compliance with the applicable Municipal Noise Control By-law. If such a by-law does not exist, the Ministry of the Environment's "Model Municipal Noise Control By-law" (17) may be used as guidance.

Also, the standby power system should be located so that contaminant levels at the nearest point of impingement due to stack emissions are in compliance with the requirements of the Environmental Protection Act.

SECTION 13

13.0 INSTRUMENTATION AND CONTROL

13.1 GENERAL

The purpose and objectives of instrumentation and control are to produce continuously high quality potable water in an economical manner in terms of manpower and resources used, and to control key functions to maintain a smooth operation. Additionally, instrumentation can provide operating data on important aspects of plant operations such as water quality, chemicals used, etc.

The requirements for instrumentation and control will be highly dependent on the size of plant and the type of process employed. In general, instrumentation and controls should be provided to allow safe and efficient operation of all parts of the plant, with minimum operator effort, and all automatic controls should be provided with manual back-up systems.

Where some parts of the plant may be operated or controlled from a remote location, local control stations should be provided and shall include the provision for preventing operation of the equipment from a remote location.

13.2 INSTRUMENTATION SYSTEMS

The minimum requirements are as follows:

- finished water chlorine residual: continuous monitoring and recording with alarms capable of transmission to a remote location;

- finished water turbidity: continuous monitoring and recording;

Additional systems which should be considered are shown below. Recommended systems are identified as 'REC'.

SYSTEM	INDICATE	RECORD	TOTALIZE
Raw water temperature	REC		N.A.
Raw water flow	REC		REC
Raw water pH (if adjusted)	REC		N.A.
Raw water turbidity	REC	REC	N.A.
Filter rate of flow	REC	REC	N.A.
Filter loss of head	REC	REC	N.A.
Filter effluent turbidity	REC		N.A.
Washwater flow	REC		N.A.
Clearwell/Reservoir level	REC		N.A.
Treated water pH (if adjusted)	REC		N.A.
Treated water fluoride (if adjusted)	REC	REC	N.A.
Treated water flow	REC		REC
Treated water pressure	REC		N.A.
Water quality - process sensitive parameters	REC		N.A.

Where single analysers, or primary devices, are used on a time-shared basis for monitoring multiple points, e.g. a single turbidimeter for multiple filter effluent samples, the rate of sample flow to the instrument should be sufficient to give a true

indication of the sample value within the time allotted to that sample.

The designer should ensure that samples taken are fully representative of the conditions, e.g., proper mixing of chemicals has occurred, and where analysers are part of an automatic control loop (see Section 13.3) the system lag time should be minimized to avoid hunting or other instabilities.

13.3 CONTROL SYSTEMS

The type of control provided at a water treatment plant can vary from the most simple manual control without any automatic function, through common systems such as semi-automatic control where operators may start or stop pumps and all other plant functions are automatic, to complex systems based on distributed intelligence micro-processors with central supervisory computers.

In selecting a control system from such a wide range of options, the designer should consider the following factors:

- Manual control systems are simpler to maintain and repair than automatic systems, and are lower in initial cost.

- sophisticated control systems require stilled maintenance.

- automatic control systems provide a higher quality product consistent with lower labour costs.

- automatic systems are only as reliable as the primary sensing devices used.

- automatic systems must be designed to manage any set of conditions which may occur.

- manual system initial low costs may be outweighed by high labour, chemical, and energy costs incurred by poorer process control

- micro-electronic systems are more reliable than mechanical systems (no moving parts) and are becoming progressively cheaper.

Overall, the designer should select a control system which will succeed with plant operating staff, be likely to continue to operate efficiently, and provide a fully effective method of producing continuously high quality water at low overall cost.

SECTION 14

14.0 LABORATORY FACILITIES

It is recommended that sample lines be brought to the laboratory from the various stages of the treatment process; for example, raw water, settled water, filtered water, and finished water. These sample lines should run continuously to provide representative samples, and be kept as short as possible.

In general, sufficient space and equipment should be provided so that all necessary testing may be conducted which is required for proper treatment process operation and control.

As a minimum, it is recommended that equipment be provided which would meet the requirements detailed in "Standard Methods" for the determination of hydrogen ion concentration in the pH range 4-10, chlorine concentration, both free and combined, in the range 0-5 mg/L, turbidity over the range normally found for the raw water source, but also capable of turbidity determination in the 0-1 NTU range. Where the process treatment involves reduction of raw water colour, equipment should be provided to determine this colour both in the raw water and finished water quality ranges. Where fluoridation is practised, equipment shall be provided for the determination of residual fluoride. In addition, it may be useful to provide certain specific ion electrodes which are compatible with the pH meter.

Where coagulants are used, and sedimentation is part of the treatment process, equipment for jar testing should be provided to aid in establishing the optimum coagulant dosage for changing water conditions.

Sufficient glassware and general reagents should be provided to conduct all the required analyses, and adequate supplies of treatment chemicals and alternates, e.g. various polyelectrolytes, should be provided for laboratory-scale investigations.

Consideration should be given to providing voltage stabilization in the electrical services to the laboratory, since instrumental analysis requires a fairly constant voltage for proper operation.

The minimum linear bench space should be three metres, including a wash-up sink. Where space is available, or the size of the laboratory permits, the space should be divided into "dry" and "wet" areas so that sensitive equipment is not subjected to undesirable conditions.

At larger treatment plants, the designer should consider providing pilot scale facilities with sufficient flexibility to alter coagulation flocculation, filtration, and other process operations, to assist in determining optimum plant operating conditions.

SECTION 15

15. SAFETY

The designer is referred to the "Safety Manual" (18) prepared by the Ministry of the Environment; the Occupational Health and Safety Act, 1978 and Regulations for Construction Projects (19); the Occupational Health and Safety Act, 1978 and Regulations for Industrial Establishments (20); the National Fire Code of Canada (NRC Publication 14987) (21); and the booklet "Hazardous Chemicals Data 1975" (10) (NFPA 49) (41).

Equipment suppliers and chemical suppliers should also be contacted regarding particular hazards of their products, and the appropriate steps taken in the facility design to ensure safe operation.

SECTION 16

16. PERSONNEL FACILITIES

The necessity for personnel facilities will be largely dictated by the number of operating staff required, and the time periods during which the plant is manned.

As a minimum it is recommended that provision be made for storage lockers, preferably two for each employee (one for work clothes, one for clean clothes), and a washroom with shower. As the size of the plant and number of staff increases, there will be a requirement to provide more locker space, possibly in a separate change room; a lunch room which should be of adequate size to serve as a meeting or instruction room for plant staff; and a suitable office for plant supervisory staff and record keeping.

Whenever possible, these personnel facilities should be separated from the plant facilities, but with convenient access to the plant.

SECTION 17

17. BUILDING SERVICES

Adequate heating facilities of a safe type should be provided with control levels depending on the type of area being heated. In many areas of the plant, sufficient heat need only be provided to prevent freezing of equipment or treatment processes.

Buildings should be well ventilated by means of windows, doors, roof ventilators, or other means. All rooms, compartments, pits, and other enclosures below grade which must be entered should have adequate forced ventilation provided when it is necessary to enter them.

Rooms and galleries containing equipment or piping should be adequately heated, ventilated, and dehumidified, if necessary, to prevent undue condensation. Switches should be provided which would conveniently control the forced ventilation.

Buildings should be adequately lighted throughout by means of natural light, artificial lighting facilities, or both. Control switches where needed should be conveniently placed at each entrance to each room or area.

It may be advantageous to provide intercom systems between the control centre and other buildings or locations throughout the plant site. Public telephone service should be provided to the control centre as a minimum. Empty conduit systems may also be provided for future telephone and/or intercom lines.

Power outlets of suitable voltage should be provided at convenient spacing through plant buildings to provide power for maintenance equipment, extension lighting, etc. Power outlets located outside buildings may be advantageous.

Potable water service to most buildings will also be required. Reference should be made to Section 10 for requirements relating to back-flow prevention for potable water supplies.

SECTION 18

18. ENERGY CONSERVATION

Typically, over 75% of the energy expenditures in water treatment plants result from high lift pumping. It is assumed that all necessary steps have been taken during the design of the distribution system immediately served by the high lift pumps, and in the selection of the high lift pumping units themselves, to minimize this energy consumption.

Up to 10% of the total energy consumed in plant operations is expended on heating and lighting. This category probably includes those items most amenable to energy conservation. The designer should carefully examine work environment items such as heating, lighting, ventilation, and air conditioning, both in the design and operation stages to ensure that heating and lighting levels are matched to job requirements without safety hazards. Time delayed switches on lights should be provided in those parts of the plant rarely used, and heating requirements should be reduced not only by adequate insulation in exterior walls, but also be separating areas which need different temperature levels; for example, providing walls between filter operating galleries and filters.

Valuable guidance on energy requirements, and design and operating conservation measures may be found in the following documents:

"Energy Requirements for Water Treatment Systems", (22) M.O.E. Research Paper S2043

"Optimization of Energy Allocation in Water and Wastewater Treatment Systems", (23) M.O.E. Research Report W65

"Industrial Energy Conservation Handbook", (24) Energy, Mines & Resources

"Canadian Code for Energy Conservation in New Buildings", (25) National Research Council of Canada.

APPENDICES

APPENDIX A: GLOSSARY OF SYMBOLS AND ABBREVIATIONS

A.A.S.H.O. American Association of State Highway Officials

A.S.T.M. American Society for Testing Materials

A.S.U. areal standard units

A.W.W.A. American Water Works Association

°C degree Celsius

C.G.S.B. Canadian Government Standards Bureau

C.S.A. Canadian Standards Association

d day

G flocculation energy gradient

Gt	flocculation energy
H.C.U.	Hazen colour unit
h	hour
kPa	kilopascal
kW	kilowatt
L	litre
m	metre
m^2	square metre
mA	milliamp
min	minute
mm	millimetre
N.T.U.	Nephelometric turbidity unit
P.V.C.	polyvinylchloride

APPENDIX B: RECOMMENDED METRIC UNITS

air supply (filter wash)	- $m^3/m^2.h$	cubic metres per square metre filter area per hour
	- m/h	Metres per hour
area	- m^2	square metres
concentration	- mg/L	milligrams per litre (dilute)
	- %	per cent (concentrated e.g. sludge)
depth	- m	metres
detention time	- minutes (short) : hours (long)	
design capacity	- m3/d	cubic metres per day
filter wash quantity	- m^3/m2	cubic metres per square metre of filter area
filter wash rate	- m/h	metres per hour (equiv. $m^3/m^2.h$)
filtration rate	- m/h	metres per hour (equiv. $m^3/m^2.h$)
flow rate	- L/s	litres per second
per capita flow	- L/cap.d	litres per capita per day

pipe size	- mm	millimetres
power	- kW	kilowatt
pressure	- kPa	kilopascal (positive)
	- mm Hg	millimetres of mercury (negative)
surface overflow rate	- m/h	metres per hour (equiv. $m^3/m^2.h$)
solids loading	- kg/h	kilograms per hour (sludge t reatment)
temperature	- °C	degrees Celsius
underflow velocity	- m/min	metres per minute
velocity	- m/s	metres per second
volume	- L	litres (small)
	- m^3	cubic metre (large)
weir overflow	- m/min	metres per minute (equiv. $m^3/m^2.min$)

APPENDIX "C"

BIBLIOGRAPHY

1. A guide on Applying for the Approval of Water Works. January 1978. Environmental Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

2. Ontario Drinking Water Objectives. Pollution Control Branch, Ontario Ministry of the Environment. Revised 1982.

3. Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems, and Storm Sewers May 1979. Environmental Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

4. Water Quality Criteria for Great Lakes Waters to be used as Municipal and Industrial Water Supplies. Environment Canada Scientific Series #43 report. 1975.

5. Occurrence of Frazil Ice in Intakes. Ontario Ministry of the Environment Research Report W43, 1973.

6. Operational Variables and Limitations of Direct Filtration.
Ontario Ministry of the Environment Research Report W54, 1975.

7. Status of Direct Filtration. AWWA Report, July 1980.

8. Storage and Use of Chlorine. Ontario Ministry of Labour.

9. Dangerous Properties of Industrial Materials.
N. Irving
Sax. Reinhold Book Corp. 3rd ed. 1968, 4th ed. 1975, 5th ed. 1979.

10. Hazardous Chemicals Data 1975. U.S. National Fire Protection Association booklet #49.

11. Minimizing and Recycling Water Plant Sludges. AWWA 1973 seminar proceedings.

12. Water Treatment Waste Disposal. AWWA 1978 Seminar proceedings.

13. Treatment and Disposal of Water Plant Wastes in Ontario.
Canada-Ontario Agreement Report #77, 1978.

14. Recommended Guidelines for Small Groundwater Supply Systems for Residential Development. August 1981 revision, joint Municipal Engineers Association-Ontario Ministry of the Environment publication.

15. Standard Specification for Diesel Engine Generator Sets
Ontario Ministry of the Environment Specification #2, Revised June 1981.

16. Standard for Protected Steel Underground Tanks for Flammable and Combustible Liquids. U.L.C.-S603.1-1977. Underwriters' Laboratory, Canada.

17. Model Municipal Noise Control By-law. Ontario Ministry of the Environment, 1978.

18. Safety Manual. Ontario Ministry of the Environment, 1969.

19. The Occupational Health and SafetY Act, 1978 and Regulations for Construction Projects.

20. The Occupational Health and SafetY Act, 1978 and Regulations for Industrial Establishments

21. The National Fire Code of Canada. NRC publication 14987.

22. Energy Requirements for Water Treatment Systems. Ontario Ministry of the Environment Research Paper S2043, 1976.

23. Optimization of Energy Allocation in Water and Waste Water Treatment Systems. Ontario Ministry of the Environment Research Report W65, 1977.

24. Industrial Energy Conservation Handbook. Energy, Mines, and Resources.

25. Canadian Code for Energy Conservation in New Buildings.
National Research Council of Canada.

SEWAGE

GUIDELINES FOR THE DESIGN

OF

SEWAGE TREATMENT WORKS

JULY 1984

THE HONOURABLE
JIM BRADLEY
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ACKNOWLEDGEMENTS

The original edition of these guidelines, issued in 1980 was prepared by Simcoe Engineering Group Limited, Consulting Engineers, Pickering, Ontario with the assistance of staff of the Ministry of the Environment. Also, other Consulting firms participated in the development of these Guidelines including Proctor and Redfern Limited, Gore and Storrie Limited, R.V. Anderson Limited and MacLaren Engineers Inc.

The revisions and expansions to the original Guidelines were undertaken by Simcoe Engineering Group Limited with the assistance of staff of the Ministry of the Environment in 1984.

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SECTION 1

GUIDELINES FOR THE DESIGN OF SEWAGE TREATMENT WORKS

1.0 INTRODUCTION

These guidelines have been prepared to document the desirable ranges, and the normal minimum or maximum acceptable levels for the various design parameters used in the design of municipal sewage treatment plants requiring approval by the Ministry of the Environment.

A complete documentation of all parameters relating to sewage treatment plant design is, of course, beyond the scope of these guidelines, but an attempt has been made to touch upon the parameters of greatest importance from process and reliability standpoints.

By issuing these guidelines, it is not the intention of the Ministry of the Environment to stifle innovation. Where the designer can show that alternate approaches can produce the desired results, such approaches will be considered for approval.

Wherever possible, designers are encouraged to use actual data derived from sewage treatment plant flow records, operational studies, etc. rather than use arbitrary design parameters. This is particularly important with sewage treatment plant expansions where the designer may want to use hydraulic and/or organic loading rates in the upper levels of the acceptable loading ranges, or where the designer proposes to deviate from recommended design parameters.

The mention in the following text of specific documents and reports is not intended to imply that these represent the sole, or most highly regarded sources of information. They may, however, be regarded as a starting point for the designer who may wish to use these documents, in concert with his own experience, to complete his design.

Designers are advised to familiarize themselves with the requirements of all legislation dealing with sewage treatment works, their associated equipment and labour safety requirements. Appendix F includes a listing of some of the most commonly referred to legislation, guidelines and other references.

The definition of abbreviation and symbols used in this guideline may be found in Appendix A.

1.1 MOE LEGISLATION

The Environmental Assessment Act, (EAA), and the Ontario Water Resources Act, (OWRA), are the two statutes, administered by the MOE, which have application to sewage treatment works.

1.1.1 Environmental Assessment Act

The EAA applies to all provincially and municipally funded/built sewage treatment works. At this point in time the EAA does not apply to sewage treatment works of the private sector. The basic assumption at the outset of planning should be, that where the Province or a municipality is involved in a sewage treatment work, (equity partner, a party to construction

contracts and/or operating agreements), the work is subject to the EAA. However, it may be the case that a particular sewage treatment work may be exempt from the EAA by regulation or exemption order. Information on the status of a sewage treatment work under the EAA can be obtained from the MOE Environmental Assessment Branch.

Sewage treatment works that are subject to the EAA cannot be proceeded with until the requirements of the Act have been satisfied. It is important to note that the EAA specifically states that for undertakings subject to the Act:

"(a) a licence, permit, approval, permission or consent that is required under any statute, regulation, by-law or other requirement of the Province of Ontario, an agency thereof, a municipality or a regulatory authority, in order to proceed with the undertaking shall not be issued or granted; and

(b) if it is intended that the Province of Ontario or any agency thereof will provide a loan, a guarantee of repayment of a loan, a grant, or a subsidy with respect to the undertaking, the loan, guarantee, grant or subsidy shall not be approved, made or given,

(c) unless, the environmental assessment has been submitted to and accepted by the Minister; and

(d) the Minister has given approval to proceed with the undertaking."

This prohibition does not apply to feasibility studies, including research, or any action necessary to comply with the EAA, e.g., site investigation and design work pursuant to the preparation of an environmental assessment.

The EAA approval must come before approvals under other acts, including any approvals that may be required under the OWRA.

In operational terms, this will mean that all technical approvals will come after the acceptance of the environmental assessment and the approval to proceed with the undertaking has been obtained. With new projects, the need to obtain specific "technical approvals" may be made a condition of the approval to proceed with the undertaking. In the case where the project presently exists, the renewal of licenses and permits should only proceed if a review indicates that the present and future operations are not substantially different from the original application. Existing regulations exempt projects at existing sewage works which are for the express purposes of maintaining or operating a work as originally proposed at the time of its establishment. Where an alteration modifies the original plant, significantly, the provisions of the EAA will have to be met.

A sewage treatment work can obtain approval under the EAA on either a "specific" (sometimes termed as "individual") or a "class" basis.

A "specific" ("individual") undertaking is one put forward by the proponent based on decision making process which has already been carried out. The environmental assessment document is a description of that process. Controversial major new sewage treatment works are likely to fall into this category.

A "class" undertaking is one in which the proponent asks for approval of an undertaking which consists of a program or class of activities. The class EA presents an evaluation of the class of activities, their alternatives and effects upon the environment. The class EA also prescribes the decision making process which will be utilized in the future to plan and develop projects which fit the description of the undertaking of the class.

The fundamental difference between the two types of EAs and their undertakings is therefore this: An individual EA deals with a special undertaking in which the time when, and the place where, the project is to be carried out are specified in environmental assessment. A "class" EA deals with a category of projects wherein time and place are not known.

The use of "class" environmental assessment is a method of dealing with certain types of projects which have important characteristics in common. Such projects occur frequently and have a generally predictable range of effects, which, though significant enough to require environmental assessment, are likely to cause only relatively minor effects in most cases.

For example, the MOE has prepared two class environmental assessments dealing with expansions and upgradings to existing sewage systems and water systems. See references [61], [62] and [63]. The Municipal Engineers' Association, acting on behalf of municipalities, has prepared a class environmental assessment dealing with municipal sewage and water system projects.

The EAA, by allowing flexibility in the way an undertaking is defined, makes it possible to deal with such projects without the necessity of passing each individual project through the formal review and decision making procedure under the Act, provided that an acceptable planning process is followed. However, the proponent must justify, the choice and process of the class EA approach.

Again, a class EA describes the planning process to satisfy the requirements of the Act which the proponent proposes to follow each time a project within that class is undertaken in the future.

The class EA document is submitted and processed like any other EA. This includes the possibility of a hearing before the Environmental Assessment Board. Once the undertaking dealt with in the class EA document has received approval, and provided that the proponent complies with the approval and any terms and conditions specified in it, projects falling within the class can proceed from planning through implementation, with no further formal applications required under the EAA.

A class environmental assessment describes the characteristics, e.g., maximum or minimum size specified standard designs, etc., of projects which are included in that particular class. The method by which a project is tested at various stages of planning and development to see whether it still conforms to the description of the category of projects covered by the approval given the class EA is described. If a project does not conform it is dealt with by

other means, e.g., a different class EA, or a specific EA, or perhaps it is not significant enough to be subject to the Act.

A class EA describes the mechanism provided for the "elevation" or "bumping up" of a particular project from the class EA category to that of a specific environmental assessment. The document explains in what circumstances these "bumping" events occur. A bump-up might occur if during the course of planning, a particular project within the description of Class EA, appeared to involve severe adverse environmental effects and/or significant public controversy. The procedure and responsibility for determining whether such circumstances exist is set out as part of the planning process, described in the class EA.

The preceding described the two methods whereby a project that is subject to the Act can meet its environmental assessment responsibilities through approvals given to either individual EAs or Class EAs. It is anticipated that the majority of sewage works projects will follow this route. However, this approach will not be appropriate for certain projects (projects which must be implemented immediately to correct emergency situations, and projects which are simply too insignificant to warrant application of the Act). Proponents of these forms of projects should satisfy their environmental assessment responsibilities by obtaining orders <u>exempting</u> the projects from the Act.

Section 29 of the EAA provides for the granting of exemption orders by the Minister of the

Environment, with the approval of Cabinet. Three points must be considered when contemplating the route of exemption. First of all, the Minister is required to give his reasons for determining why it is in the best interests of the environment to grant an exemption. These reasons must appear in the text of the exemption order, which is a public document. Secondly, although an exemption is granted by order of the Minister of the Environment, the order may only be granted with the approval of Cabinet. Finally, the approval of exemptions are usually granted with conditions which seek to minimize the effect that the exempted undertaking will have upon the environment.

Applications for exemption must be made in writing to the Minister of the Environment. The application must accurately describe the undertaking for which exemption is being sought, the damage or interference which would be incurred if the project was not exempted, the reasons why the environment would be better served were the exemption to be granted, and finally, any conditions which should be placed on the approval to minimize the adverse effects upon the environment which may result from the undertaking. Criteria which may be sighted to justify a request for exemption could include the following: existence of a public health emergency situation, the insignificance of the effects upon the environment associated with the undertaking, or the extent to which the undertaking has been developed through an acceptable planning process involving adequate public participation.

It is suggested that proponents of undertakings for which exemptions are being sought discuss drafts of exemption applications with the staff of the Environmental Assessment Branch before the exemption applications are formally submitted to the Minister. It is hoped that most exemption requests will, when formally submitted, be suitable for adoption without significant word changes.

Before leaving this topic, it should be pointed out that an independent body has been established to hear, upon referral from Cabinet, matters pertaining to exemption order requests. A review by this body would typically involve an assessment of the rationale for exemption put forward by the proponent, including the adequacy in which the conditions on the approval of the order would minimize the adverse effects that the undertaking would have on the environment.

The foregoing is a general introduction to the concept of approaching the requirements of the EAA. Reference should be made to the MOE publication "General Guidelines for the Preparation of the Environmental Assessments", January, 1981.

1.1.2 The Ontario Water Resources Act

The general information required and procedures to follow in applying for approval under the OWRA are outlined in MOE Publication "A Guide for Applying for The Approval of Sewage Works".

As outlined above, approval under the EAA must be obtained prior to a project receiving approval under the OWRA. It should be pointed out that one of the conditions of approval of MOE's class environmental assessment for upgrading or expansion to existing sewage systems requires that all sewage works proceeding under the approval must, as part of the process of planning a project, obtain approval under the OWRA. Similarly, it is expected that new sewage treatment works, seeking approval as individual environmental assessments under the EAA will have as one of their conditions of approval, the requirement that the project obtain approval under the OWRA.

1.2 HEARING REQUIREMENTS

A critical component of the planning process for the development of a sewage treatment work is the opportunity for, and role of, any hearings which may be held. For any particular sewage treatment work, depending on the circumstances, there are a number of hearing opportunities under various statutes of the Province.

A project, seeking EAA approval may be the subject of a hearing, before the Environmental Assessment Board, under Section 12 or 13 of the EAA.

In the past, sewage treatment works may have been subject to hearings under a variety of sections of the OWRA. With the application of the EAA, sewage treatment work projects which

were subject to both the EAA and the OWRA might have been subject to hearings under both statutes. This situation might have subjected proponents of such works to lengthy, repetitive and potentially expensive hearing exercises.

Section 34 of the EAA provides a mechanism whereby the Minister may issue an order which removes the requirement for a hearing under either the EAA or the OWRA, in those cases where hearings under both acts are required.

In association with the approval of the class EAs for provincial and municipal sewage projects, the Minister has issued orders under Section 34 of the EAA. The orders waived the requirements for hearings under the OWRA. Thus the only hearings which may be held will be those required under the EAA pursuant to applications for approvals of individual EAs.

Sewage treatment works may also be subject to other planning-oriented legislation. For example, a project may be subject to the Planning Act, The Municipal Act, The Ontario Municipal Board Act, The Expropriations Act, Niagara Escarpment Planning and Development Act, or The Parkway Belt Planning and Development Act. Hearings may be involved in these statutes and, where a hearing is required under the EAA, the situation of overlapping and possibly conflicting hearings arises. The Consolidated Hearings Act, 1981, has been developed and is now in place to streamline and simplify the process of application, review, hearing,

decision and appeal for projects subject to hearings under a variety of planning- oriented legislation. The Consolidated Hearings Act, 1981, speeds up the approval process by allowingfor the decision making aspects of the various procedures of Provincial legislation to be conducted in one forum where all views, alternatives, and advantages of a particular project can be analyzed.

1.2.1 Consolidated Hearings Act

For more information on the Consolidated Hearings Act, 1981, contact the Environmental Assessment Branch.

1.3 EFFLUENT REQUIREMENTS

Before the design of sewage treatment works can be initiated, the designer must determine the effluent quality criteria which the sewage treatment works must achieve.

The effluent quality criteria will be determined in accordance with MOE policy 08-01. In certain instances the criteria will require calculation on a case-by-case basis to ensure a quality of water in the receiving stream consistent with the Ministry's water quality objectives. These water quality objectives are contained in the publication "Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment" Ontario Ministry of the Environment, November 1978 [4]. For further information on effluent quality requirements, the designer should contact the Regional Office of the Ministry of the Environment.

The above mentioned effluent requirements will be incorporated into Certificates of Approval issued under Section 24 of the Ontario Water Resources Act and will specify both waste loadings and concentrations.

SECTION 2

2.0 SITE CONSIDERATIONS

2.1 SEWAGE TREATMENT PLANT LOCATION

Some of the factors which should be taken into consideration when selecting a new sewage treatment works site or contemplating an extension to an existing facility are as follows:

a) adequacy of isolation from residential areas and land use surrounding plant site (see Appendix B);
b) prevailing wind directions;
c) susceptibility of site to flooding;
d) suitability of soil conditions;
e) adequacy of site for future expansion and/or provision of additional treatment stages;
f) suitability of site with respect to access to receiving body of water or other means of effluent disposal;
g) acceptability of site with respect to sludge disposal options on site or access to sludge disposal/utilization areas off-site.

In regard to land use, the Ministry of the Environment has issued "Guidelines for Compatibility Between Sewage Treatment Facilities and Residential Land Uses" (See Appendix B).

2.2 GENERAL PLANT LAYOUT

The general arrangement of the plant within the site should take into account the subsurface conditions and natural grades to provide the

necessary facilities at minimum cost. The susceptibility of the site to flooding should be investigated and, if necessary, measures taken to prevent flooding damage as may be directed by the Conservation Authority or the Ontario Ministry of Natural Resources.

In the layout of the plant, the designer should orient the buildings to provide adequate allowances for future linear expansions of the various treatment sections and orient the plant so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, misting and freezing problems and energy consumption. The plant layout should also allow for the probability of snow drifting, with entrances, roadways and open tankage located so that the effect of snow drifting on operations will be minimized.

It is not recommended that construction of any of the facilities be in close proximity to a shore line, except where this is unavoidable. Suitable measures must be taken to adequately protect the structures from the effects of wave action and shore erosion, and the designer is referred to "Ice Piling on Lakeshores", Environment Canada, Scientific Series No. 35, 1974 [6].

Within the constraints mentioned above, the designer should work towards a plant layout where the various processing units are arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludges, or chemicals from one module to the next, and also to arrange the

plant layout to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

Where site roadways are provided for truck access, the road design should be sufficient to withstand the largest anticipated delivery or disposal vehicles with due allowance for vehicle turning and forward exit from the site.

In order to avoid the dangers of high voltage lines crossing the site, it is suggested that a high voltage pole be located at the property line. Depending on the distance from the terminal pole to the control building, the step-down transformer would be located at the terminal pole or adjacent to the control building. If the distance between the terminal pole and the building is excessive, the transformer should be located adjacent to the building. Then the high voltage connections should be brought by underground cable to the pothead at the transformer. In this way, the primary and secondary terminals of the transformer are fully enclosed and no fence is required around the transformer.

Sewage treatment works sites must be adequately fenced and posted to prevent persons from obtaining unauthorized access. The perimeters of open tankage or excavations should be adequately fenced and posted to prevent persons from obtaining unauthorized access. The perimeters of open tankage or excavations should be adequately safeguarded.

2.3 PROVISION FOR FUTURE EXPANSION

In addition to the considerations of site size required to physically accommodate future treatment plant expansions, it is necessary for the designer to allow for the future expansion possibilities and/or process changes in his design.

For instance, on-site sewage pumping stations should be designed such that their capacity can be increased and/or parallel facilities constructed without the need for major disruption of the plant's operation. Wherever possible, the layout and sizing of channels and plant piping should be such that additional treatment units can be added in future or increases in loading rates can be accommodated hydraulically. Similarly, the location of buildings and tankage should consider the location of the next stages of expansion.

Within buildings, space should be provided for replacement of equipment with larger capacity units. This is particularly important with equipment such as pumps, blowers, boilers, heat exchangers, etc. Adequate working space must be provided around equipment and provision made for the removal of equipment for replacement, or major maintenance operations.

In sizing inlet and outlet sewers, the ultimate plant capacity should be considered. Provided that problems will not occur with excessive sedimentation in the sewers, these sewers should be sized for the ultimate condition. With

diffused outfalls, satisfactory port velocities can often be obtained by blocking off ports which will not be required until subsequent expansion stages.

SECTION 3

3.0 PLANT CAPACITY

In the Canada-Ontario Agreement Research Report No. 15, "Identification of Problem Areas in Water Pollution Control Plants" [7], one of the most common problems with various plant components was that of hydraulic overloading. The report indicated that, of the plants constructed in Ontario in the preceding five years 33 per cent were hydraulically overloaded. The report recommended that " a re-evaluation of the methods used in projecting dry-and-wet-weather flows should be considered". Research Reports No. 11 and 55, Volumes I and II, "To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies with Reference to Flow Variations" [8] also deal with the typical flow variations experienced in sewage treatment plants and suggest methods to be used to size flow equalization facilities. The cost-benefits of such facilities are also covered.

The following sections discuss the recommended approach to arriving at the required design capacity for the various components of a sewage treatment plant.

3.1 DESIGN PERIOD

Factors which will have an influence on the design period of sewage treatment works include the following:

- population growth rates;
- prevailing financing interest rates;
- inflation rates;
- ease of expansion of facilities;

- time requirements for design and construction of expansion.

Wherever possible, sewage treatment plants should be designed for the flows expected to be received 20 years hence, under normal growth conditions. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "on-line" when needed.

3.2 SEWAGE FLOWS AND STRENGTHS

Wherever there are existing sewers and/or existing sewage treatment plants, the flow rates and sewage strengths should be determined, preferably in both wet-and-dry weather conditions.

Analytical techniques should then be used to estimate the following:

- average and peak normal wastewater flows[1]. in the design year;
- average infiltration and inflow in the design year;
- peak infiltration in the design year;
- peak inflow in the design year;
- peak infiltration/inflow in the design year.

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1. Normal wastewater flows are the flow components actually generated within the buildings serviced by the sewer system, and are exclusive of any extraneous flows.

A detailed discussion of the techniques involved in determining the above flow components can be found in the EPA Publication "Handbook for Sewer System Evaluation and Rehabilitation", EPA 430/9-75-021 [9].

Where it is found that sewage strengths vary significantly over the year due to excessive infiltration/inflow, population variations and/ or seasonal changes in industrial or commercial operations, estimates should be made of the expected average, maximum, and minimum BOD and suspended solids concentrations in the sewage for each month of the year. If nitrification is required, short-and-long-term variations in ammonia and total Kjeldahl nitrogen concentrations should also be estimated.

During investigations to determine peak extraneous flows, it may be found that such flows are excessive and that measures should be taken to reduce the extraneous flows rather than provide flow equalization and/or treatment facilities necessary to accommodate the excessive flows at the sewage treatment works. It is often difficult to determine when measures to reduce infiltration will be cost-effective, but experience in the United States has indicated that if infiltration, based upon the highest weekly average within a 12 month period, is less than 0.14 (L/mm.d)/m (litres per millimetre of pipe diameter per day per linear metre of sewer length) rehabilitation of the sewer system will not be economical.

In many instances, due to the lack of existing sewer systems, it will be impossible to base

estimates of sewage flows and strengths upon actual field measurements. In such cases, the flow records and sewage strengths of similar serviced communities may provide data upon which estimates can be based. The Utility Operations staff of the Ministry of the Environment may be contacted for flow rate and sewage characteristics of sewage plants operated by the Ministry of the Environment. For municipally-operated plants, the same data may be obtained from the municipality, or from the Ministry of the Environment's Municipal and Private Abatement Section.

If it is found necessary to use purely empirical data to estimate sewage flows and strengths, it is suggested that no less than 225 L/cap.d be used for average domestic wastewater flows, exclusive of extraneous flows. Extraneous flows, including infiltration and inflow, will vary greatly from municipality to municipality, but it is suggested that no less than 0.1 (L/mm.d)/m (litres per millimetre of sewer diameter per day per linear metre of total sewer system length including service connections) be used for peak extraneous flows. (See the Appendix F, for methods of establishing the extraneous flow allowances). To estimate peak sewage flows, the average domestic flow rate should be multiplied by the Harmon factor, then the peak extraneous flows should be added. Industrial and commercial sewage flow rates should be calculated separately and added to the above sewage flow rates. Typical organic loading rates for domestic sewage are 75 and 90 g/cap.d for BOD_5 and SS, respectively.

3.3 DESIGN CAPACITY OF VARIOUS PLANT COMPONENTS (WITHOUT FLOW EQUALIZATION)

In general, all components of mechanical sewage treatment plants should be hydraulically capable of handling the anticipated peak sewage flow rates without overtopping channels and/or tankage. From a process point-of-view, however, the design of various sections of sewage treatment plants should be based upon the following hydraulic, organic and inorganic loading rates:

Sewage Pumping Stations
- peak flow rate.

Screening
- peak flow rate.

Grit Removal
- peak flow rate, peak grit loading rate.

Primary Sedimentation
- peak flow rate, peak suspended solids loading rate.

Aeration (without nitrification)
- average diurnal BOD_5 loading rate usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Daily or seasonal variations in domestic and/or industrial BOD loading rates should be taken into consideration. Except for short detention treatment systems such as contact stabilization or

high rate processes, hydraulic detention time is seldom critical.

Aeration (with nitrification)

- average diurnal BOD_5 loading rates usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient BOD diurnal variations to warrant consideration. Diurnal peak flow rate and diurnal peak ammonia (total Kjeldahl nitrogen with extended aeration) loading rates must be designed for. Daily or seasonal variations in BOD_5, ammonia, total Kjeldahl (with extended aeration) and peak flow rates should also be taken into consideration.

Secondary Sedimentation

- peak flow rate or peak solids loading rate, whichever governs (in some cases, where existing peak flows are excessive and where adequate assimilation capacity exists in the receiving stream, by- passing a portion of the peak flow around the secondary treatment section, after primary treatment may be permitted).

Sludge Return

- capacity requirements will vary with treatment system (see SECTION 10).

Disinfection Systems

- average flow rate, unless downstream uses of receiving stream dictate otherwise and then detention time must be provided for peak flow rates.

Chlorine and Other Chemical Feed
- peak flow rate.

Effluent Filtration
- peak flow rate, peak solids loading rate.

Outfall Sewer
- peak flow rate.

Sludge Treatment (digestion, thickening, dewatering, incineration, etc.)
- average loading rates (hydraulic, total solids, volatile solids) unless sustained peaks are of significance to the individual treatment process.

3.4 FLOW EQUALIZATION

In reference [8], it was concluded that full equalization of diurnal sewage flow peaks can result in a small reduction in construction costs over variable flow design and in addition can result in reduced energy costs and improved treatment efficiency. Partial equalization of sewage flows was not found to be a viable alternative due to the decreased benefits with only slight savings possible with the required storage volumes.

The sizing of the storage facilities and downstream treatment units when flow equalization is used is discussed in references [8] and [39].

3.5 OUTFALL SEWERS

The proper siting and design of the plant outfall structure is important in minimizing the

impact on receiving water quality. In many cases it may be a controlling factor in ensuring protection of nearby water supplies, recreational beaches or fisheries habitat.

Outfalls should be designed and located so as to obtain the greatest possible dilution of the effluent during the periods of greatest susceptibility of nearby water uses to adverse impact.

Dilution is a product of initial mixing of the effluent with surrounding water and subsequent dispersion due to water movement.

Entrainment of ambient lake or stream water into the effluent is generally enhanced by extending the outfall away from the shore into deeper water and often by incorporating a multiport diffuser to spread the discharge over a larger area and to increase turbulent mixing. Similarly, dispersion is aided by maximizing the separation of the discharged plume from boundary effects of the shoreline and lake or streambed.

In determining the desired level of dilution to be achieved, reference should be made to the implementation procedure for defining mixing zones contained in the publication, "Water Management Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment - November 1978". Recommended procedures for predicting initial dilution and dispersion for a variety of outfall types and receiving water considerations may be obtained from the Water Resources Branch of the Ministry of the Environment.

Prediction requires knowledge of effluent concentration, discharge rates, effluent buoyancy, jet velocity, ambient current velocity, depth of water over the outfall, ambient thermal regime (vertical temperature profile) and background water quality.

For all extended outfalls, outfall capacity should be sufficient to handle not only the treated effluent but also all storm flows received at the plant so as to eliminate bypassing of untreated or partially treated flows at shore.

SECTION 4

4.0 COMPONENT RELIABILITY

4.1 COMPONENT BACK-UP REQUIREMENTS

The components of sewage treatment plants should be designed in such a way that equipment breakdown and normal maintenance operations can be accommodated without causing serious deterioration of effluent quality.

To achieve this, critical treatment processes should be provided in multiple units so that with the largest unit out of operation, the hydraulic capacity (not necessarily the design rated capacity) of the remaining units shall be sufficient to handle the peak wastewater flow. There should also be sufficient flexibility in capability of operation so that the normal flow into a unit out of operation can be distributed to all the remaining units. Similarly, it should be possible to distribute the flow of all of the units in the treatment process downstream of the affected process. In addition, where feasible, it should be possible to operate the sections of treatment plants as completely separate process trains to allow full-scale loading tests to be carried out.

With some processes such as mechanical screening or comminution, the back-up can be provided with a less sophisticated unit such as a manually-cleaned screen.

Sewage and sludge pumping units should always be provided with a back-up pump of equal capacity to the largest duty pump. In certain instances, particularly with sludge pumps, one pump may

serve as a back-up for more than one set of pumps, i.e., a raw sludge pump could back-up a sludge transfer pump, etc. Standby capacity requirements for sludge return pumps will be determined on a case-by-case basis.

Depending upon the size of the sewage treatment plant and the sensitivity of the receiving waters, some unit processes may not require duplication. For instance, if the equivalent of primary treatment would be satisfactory under emergency conditions, one aeration basin may be sufficient.

Aeration systems will require facilities to permit continuous operation, or minimal disruption, in the event of equipment failure. The following factors should be considered when designing the back-up requirements for aeration systems:

- effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);

- time required to perform the necessary repair and maintenance operations;

- the general availability of spare parts and the time required to obtain delivery and installation;

- means other than duplicate equipment to provide the necessary capacity in the event

of a breakdown (for instance, using oversized mechanical aerators with adjustable weirs to control power draw and oxygenation capacity, or using two speed mechanical aerators, etc.).

Generally considerations such as the above will mean that diffused aeration systems will require a standby blower but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

Chemical feed equipment for phosphorus removal and disinfection should be provided in multiple units so that the chemical requirements can be supplied with one unit out of operation.

With sludge digestion facilities, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumpage and mixing so that one stage can be serviced while the other stage receives the raw sludge pumpage. In smaller plants, multiple primary and secondary digestion units can often be avoided by this method. When such an approach is proposed, the designer should outline the alternate methods of treatment and disposal that could be used during periods of equipment breakdown. With larger treatment plants, the provision of multiple primary and secondary digestion units can usually be economically justified. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating. They will be considered, however, where the designer can show that the

above concerns can be satisfied and that alternate means of sludge disposal can be used in the event of breakdown.

Depending upon the receiving stream sensitivity, and type of filtration equipment, and the maintenance requirements of the filter units, the provision of multiple effluent filtration may not be necessary.

With sludge handling and dewatering equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated r additional shifts in the event of equipment breakdown.

4.2 SEWAGE OVERFLOW AND BY-PASS FACILITIES

If sewage entering the treatment plant must be pumped into the treatment units, an emergency overflow for the pumping station should be provided, if it is physically possible. The purpose of this overflow is to prevent basement flooding by back-ups in the sewer system in the event of pumping station failure. Wherever possible, this overflow should be routed through the chlorine contact chamber and plant outfall sewer. If this is not possible, provision should be made for chlorination of such verflows.

The overflow elevation and the method of activation should ensure that the maximum feasible storage of the sewage collection system and wet

well will be utilized before the controlled overflow takes place. The overflow facilities should at least be alarmed and equipped to indicate frequency and duration of overflows and provided with facilities to permit manual flow measurement. Automatic flow measurement and recording systems may be required in certain cases where effluent quality requirements dictate.

To allow maintenance operations to be carried out, each unit process within the treatment plant should be provided with a by-pass facility around the unit.

Where two or more similar treatment units are considered and one unit is out of operation for repairs, the remaining units should be capable of passing the peak sewage flow rates or be provided with by-pass capacity equal to the excess hydraulic flow of the operating units.

By-pass systems should also be constructed so that each unit process can be separately by-passed, i.e., no need to by-pass more unit processes than necessary.

All flows by-passing secondary and/or tertiary treatment processes should be measured.

If a by-pass for the chlorine contact chamber is required, provisions may be necessary for emergency chlorination of flows in the by-pass channel.

4.3 STANDBY POWER

The need for standby power and the extent of equipment requiring operation by standby power must be individually assessed for each sewage treatment plant. Some of the factors which will require consideration in making the decisions regarding standby power and the processes to be operated by the standby power equipment are as follows:

- reliability of primary power source;
- number of power feeder lines supplying grid system, number of alternate routes within the grid system, and number of alternate transformers through which power could be directed to the sewage treatment plant;
- whether sewage enters plant by gravity or is pumped;
- type of treatment provided;
- pieces of equipment which may become damaged or overloaded following prolonged power failure;
- assimilation capacity of the receiving waters and ability to withstand higher pollutional loadings over short time periods;
- other uses of receiving waters.

The Ministry publication "Guidelines for the Provision of Equipment to Handle Emergency

Conditions (Power Outages) in New Sewage Works in the Province of Ontario", reference [11], requires, as a minimum, that all sewage treatment works should be required to provide at least primary treatment and chlorination, or the equivalent, for all collected sewage. Each specific installation should provide for the following considerations:

(i) means for illuminating working areas to ensure safe working conditions;

(ii) standby power source or equivalent to power pumps, motorized valves and control panels that are necessary to maintain the sewage flow through the treatment plant.

For the suggested design of Diesel standby power equipment, reference should be made to the Ministry's "Standard Specifications for Diesel Generator Sets".

Where standby power is not needed for pumpage or treatment, the designer should give some consideration to the provision of a small (typically 25 kW) generator set having sufficient capacity to provide the power for lighting and instrumentation, so that in the event of transient power outages the plant will have sufficient power available for safe operation and to maintain proper instrumentation.

It should be noted that it is standard practice when running Diesel engines to permit them to run for not less than 60 minutes to avoid sludging and other problems. For this reason the designer should give consideration to provide a standby power system with manual

start-up or with automatic start-up utilizing a timer to delay the start-up during momentary power failures which would prevent the Diesel engine from unnecessarily running for 60 minutes. Timers should also be provided to bring equipment on line in such a way that the generators will not be overloaded by the starting current requirements of motors. Similar protection will be necessary to avoid overload of the normal electrical supply on resumption of power following a power failure.

The standby power equipment should preferably be located so that it connects conveniently into the electrical distribution system of the plant, and so that it is close to other potentially noisy equipment, so that adequate acoustic measures need only be taken over small areas. Sufficient fuel storage should be provided, taking into account the historical data on length of power outages in the area, and any weather or other conditions which might preclude fresh deliveries of fuel. A minimum 450 L fuel tank should be provided for generator set capacities of up to 25 kW, and 900 L fuel tank for set capacities from 30 to 100 kW, 1135 L fuel tank for set capacities from 110 to 150 kW and 2 x 1135 L fuel tanks for set capacities from 160 to 300 kW.

Either underground or inside fuel storage tanks may be used. In considering which type to use, factors such as corrosion potential, consequences of leakage, storage volume needed, need for fuel pumps, etc. should be evaluated. For details on the requirements for underground storage tanks refer to ULC - S603.1 - 1977

"Standard for Protected Steel Underground Tanks for Flammable and Combustible Liquids", Underwriter's Laboratory, Canada.

The location of the standby power system should generally be such that site perimeter noise levels will be in compliance with the Ministry's "Model Municipal Noise Control By-law", and also located so that contaminant levels at the nearest point of impingement due to stack emissions are in compliance with the requirements of the Environmental Protection Act.

SECTION 5

5.0 PLANT HYDRAULICS

5.1 SEWAGE PUMPAGE

Raw sewage and any intermediate sewage pumping stations associated with sewage treatment works should be capable of conveying the peak sewage flow rates expected to downstream treatment units. Pumping equipment should also be designed so that downstream treatment units are not subjected to unnecessary surging. To achieve this, the necessary pumpage capacity can be provided by variable capacity, or multiple fixed capacity pumps, so that pump discharge rates will closely match the sewage inflow rate.

The Ministry of the Environment's guideline entitled, "Guidelines For The Design Of Sanitary Sewage Systems" [13] should be referred to for the recommended approaches to sewage pumping system designs.

5.2 CHANNEL FLOW

Channels must be designed to convey the initial and ultimate range of flows expected. To avoid solids buildup, the following scouring velocities should be developed in normally used channels at least once per day:

sewage containing grit	- 0.9 m/s
floc suspensions	- 0.45 to 0.60 m/s

Where the above scouring velocities cannot be obtained, channels may be aerated to prevent solids deposition.

5.3 FLOW DIVISION

Within sewage treatment plants, there will invariably be situations where flow splitting is necessary. Unless certain precautions are taken, the flow will not split in the proportions desired over the full flow range, or the flow may split properly, but the organic load will not be divided in the same proportion.

To ensure that the organic load splits in the same proportion as the flows, the suspended solids must be homogeneously dispersed throughout the liquid and the relative momentum of all particles must be approximately equal at the point of diversion. Some turbulence is therefore desirable before each point of diversion. The following methods can be used to produce homogeneity:

- mechanical mixers
- diffused aeration
- bottom entrance into splitting box
- bar racks or posts in channels
- straight section of conduit 6 to 8 diameters (or channel width) upstream of point of diversion

Flow splitting methods commonly used in sewage treatment plants are shown in Figure 5.1. For flow splitting from a channel where the hydraulic head at diversion points is different (Type III), see references [14] and [15], for the method to be used to calculate the port sizing to ensure uniform flow splitting.

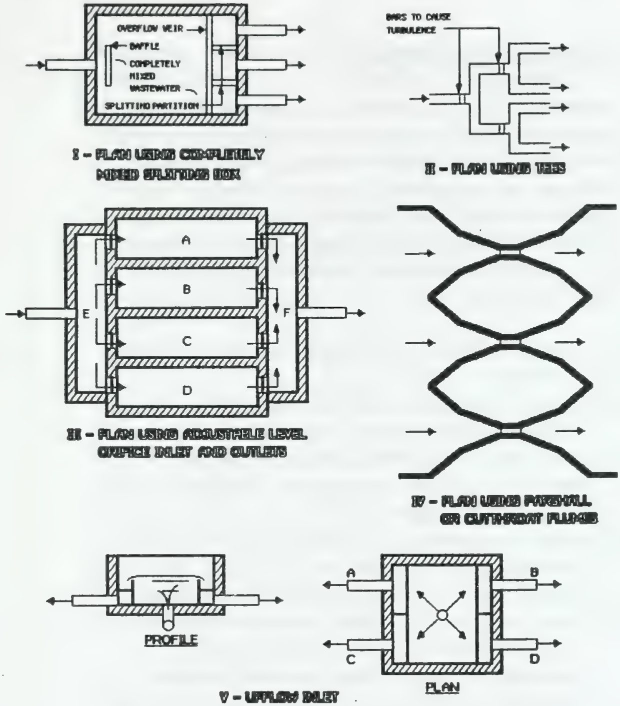


FIGURE 5.1

METHODS OF WASTEWATER FLOW DIVISION

NOTE

This information is from the EPA-625/1-77-009 Process Design Manual "Wastewater Treatment Facilities for Sewered Small Communities".

5.4 PLANT HYDRAULIC GRADIENT

The hydraulic gradient of all gravity flow and pumped waste streams within the sewage treatment plant, including by-pass channels, should be prepared to ensure that adequate provision has been made for all head losses. In calculating the hydraulic gradient, changes in head caused by all factors should be considered, including the following:

a) head losses due to channel and pipe wall friction;
b) head losses due to sudden enlargement or sudden contraction in flow cross section;
c) head losses due to sudden changes in direction, such as at bends, elbows, Y-branches and tees;
d) head losses due to sudden changes in slope, or drops;
e) head losses due to obstructions in conduit;
f) head required to allow flow over weirs, through flumes, orifices and other measuring, controlling, or flow division devices;
g) head losses caused by flow through comminutors, bar screens, tankage, filters and other treatment units;
h) head losses caused by air entrainment or air binding;
i) head losses incurred due to flow splitting along the side of a channel;
j) head increases caused by pumping;
k) head allowances for expansion requirements and/or process changes;
l) head allowances due to maximum water levels in receiving waters.

Designers are cautioned to consider the consequences of excessive or inadequate allowances for head losses through sewage treatment works. If pumpage is required, excessive head loss allowances result in energy wastage. If inadequate head loss allowances are made, operation will be difficult and plant expansion more costly. References [16], [17], [19] discuss in considerable detail the calculation of head losses in sewage treatment plants.

5.5 SLUDGE PUMPAGE

The flow characteristics of sludges will vary according to the types and concentrations of organic solids and added chemicals. Some sludges will be similar to clear water while others may have pseudoplastic flow characteristics. Sludges may also exhibit thixotropic behaviour; that is, their viscosity increases with time when in a static state, but on agitation returns to its original value.

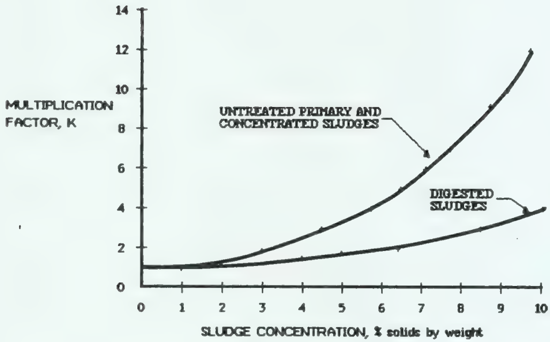
The friction losses associated with sludge pumpage applications vary greatly, from losses similar to those experienced with clean water, for dilute sludges, to losses greater by a factor of 15, or more, times clean water losses for thickened sludges. Figure 5.2 from reference [20] shows the multiplication factor to apply to the friction losses for turbulent flow for clean water to calculate the friction losses for untreated primary and concentrated sludges and digested sludge. Use of Figure 5.2 will often provide sufficiently accurate results for design, especially at solids concentrations

below 3 percent. However, as pipe length, percent total solids and percent volatile solids increase, more elaborate methods may have to be used to calculate the friction losses with sufficient accuracy. Reference [20] also outlines the procedures to follow to calculate friction losses once the sludge's density, yield stress and coefficient of rigidity are known.

With sludge pumpage, velocities of 0.9 to 1.5 m/s should be developed. For heavier sludges and grease, velocities of 1.5 to 2.4 m/s are needed. To avoid blockages, a minimum line size of 100 mm should be used.

FIGURE 5.2

APPROXIMATE FRICTION HEAD-LOSS FOR LAMINAR FLOW OF SLUDGE



NOTES :

1. Multiply loss with clean water by K to estimate friction loss under laminar conditions (see text).

2. The information on this figure has been extracted from EPA 625/1-79-011 "Process Design Manual for Sludge Treatment and Disposal", September 1979.

SECTION 6

6.0 EFFLUENT REQUIREMENTS AND SELECTION OF TREATMENT ALTERNATIVES

MOE Policy 08-01 and its related guidelines covers the effluent requirements for sewage treatment plants. Designers should consult with MOE Regional staff to determine the treatment requirements for specific proposals. The normal level of treatment for municipal sewage discharges to surface waters in the Province of Ontario is secondary treatment. On a case-by-case basis, relaxation of this treatment requirement to primary treatment will be permitted after receiving water assessment study results and other environmental, social and economic factors have been considered and a relaxation of treatment requirement has been justified.

Treatment beyond the norm of secondary requirement for various watersheds may be necessary due to limited assimilation capacity, and/or critical downstream uses being made of the receiving body of water. Many watersheds in the Province have, after extensive study, been designated as requiring higher levels of treatment.

Phosphorus removal has been implemented in the drainage areas of the Lower Great Lakes, parts of the Ottawa River System, parts of the Upper Great Lakes and throughout the inland recreational areas. In the rest of the Province, phosphorus removal is not required unless study results justify the need. The Ministry of the

Environment has a draft policy and draft guidelines which establish the capacity of plants requiring phosphorus removal and the effluent phosphorus concentrations for the various watersheds. The Ministry's Regional Office should be contacted to determine phosphorus removal requirements for sewage treatment works discharging to a particular watershed.

For certain watersheds in the Province sewage treatment plants may also be required to produce a nitrified effluent, due to either concerns with un-ionized ammonia toxicity or nitrogen-related oxygen demands in the receiving waters.

Depending upon the effluent requirements of the receiving body of water, there are a number of suitable alternative sewage treatment processes that can be considered. Table 6.1 lists the processes and the expected effluent quality produced by well operated plants treating normal strength municipal waste waters.

Other factors besides expected effluent quality which will affect the choice of treatment processes are such things as capital and operating costs, ultimate sludge disposal options, available land area, labour skills, soil conditions, need for retention of treated sewage during periods of the year, etc. Before deciding upon the sewage and sludge treatment processes, the designer should evaluate the alternatives available, in terms of overall capital and operating costs, to ensure that the most cost-effective treatment system is selected.

TABLE 6.1

SEWAGE TREATMENT PROCESSES AND TYPICAL EFFLUENT QUALITY

PROCESS	EFFLUENT PARAMETERS (mg/L)			
	TOTAL BOD_5	SS	TOTAL PHOSPHORUS (as P)	FREE AMMONIA (as N)
PRIMARY				
- Without P Removal	110	70	5.0	20
- With P Removal	90	30	1.0	20
CONVENTIONAL A.S.				
- Without P Removal	15	15	3.5	17
- With P Removal	15	15	1.0	17
- With P Removal And Filtration	10	5	0.3	17
- With Nitrification	15	15	3.5	3.0
CONTACT STABILIZATION				
- Without P Removal	20	20	3.5	17
- With P Removal	20	20	1.0	17
EXTENDED AERATION				
- Without P Removal	15	15	3.5	3.0
- With P Removal	15	15	1.0	3.0
- With P Removal And Filtration	5	5	0.3	3.0
CONTINUOUS DISCHARGE LAGOON				
- Without P Removal	25	30	6.0	
- With P Removal	25	30	1.0	
SEASONAL RETENTION LAGOON				
- Without P Removal	25	30	6.0	
- With P Removal By Batch Chemical Dosage	15	20	1.0/0.5	
- With P Removal By Continuous Chemical Dosage	25	30	1.0	
PRE-AERATION LAGOON (Aerobic - Facultative Type)				
- Without P Removal With 4-5 Days Retention Time	60	100	6.0	

NOTE :

1. The above values are based on typical raw sewage with Total BOD_5 = 170 mg/L, Soluble BOD_5 = 50%, SS = 200 mg/L, P = 7 mg/L, NH_4^+ = 20 mg/L.

To assist the designer in evaluating the energy component of the operating costs, the Province of Ontario has produced a number of publications dealing with this subject [21], [22], [23], [24], and [25] as follows:

- "Guidelines for Energy Conservation in the Design of Sewer Systems and Sewage Treatment Facilities in the Province of Ontario", October 1977;

- "Energy Requirements for Conventional and Advanced Wastewater Treatment", October 1975;

- "Optimization of Energy Allocation in Water and Wastewater Treatment Systems", M.O.E. Research Report W65;

- "Conservation Measures in Wastewater Treatment Plants, a Feasibility Study", October 1978;

- "Energy Management Program (Energy Bus), Mobile Support Unit".

Two other references prepared by the Federal Government [26] and [27] are as follows:

- "Industrial Energy Conservation Handbooks", Energy, Mines & Resources Canada;

- "Canadian Code for Energy Conservation in New Buildings", National Research Council of Canada.

In the following sections, the suggested design parameters for the various unit processes of the treatment systems are discussed.

SECTION 7

7.0 SCREENING

Coarse screens, or trash racks, should be provided as the first treatment stage for the protection of plant equipment against reduced operating efficiency, blockage, or physical damage. Coarse screening can be provided in the form of bar screens (manually or mechanically cleaned) or, comminutors (including barminutors). Due to the problems which can occur due to the recombining of comminuted or barminuted materials in downstream treatment units, the Ministry of the Environment recommends the physical removal of the coarse material from the waste flow stream rather than the use of shredding or cutting devices with reintroduction of the material to the waste flow stream.

When comminutors are used, they are commonly placed downstream of grit removal to avoid damage to the cutters caused by grit particles. On the other hand, if comminution is provided upstream of grit removal units, more efficient grit removal will be achieved. If mechanical grit removal is used, equipment protection in the form of some type of coarse screening device should be provided upstream of the grit removal facilities.

When considering which types of screening devices should be used, the following factors should be considered:

- effect on downstream treatment and sludge disposal operations;

- possible damage to comminutor or barminutor devices caused by stones or coarse grit particles;

- head losses of the various alternative screening devices;

- maintenance requirements;

- screenings disposal requirements.

Facilities for the removal, drainage, storage and ultimate disposal of accumulated screenings must be provided when manually or mechanically cleaned screens are used.

SECTION 8

8.0 GRIT REMOVAL

Grit removal is required in advance of treatment units to prevent the undue wear of machinery and the unwanted accumulation of solids in channels, settling tanks and digesters. See Table 8.1 for the volume of grit removed by Ministry operated plants throughout the Province of Ontario.

Grit removal is normally accomplished by grit channels, detritus tanks or aerated grit tanks. Grit removal can also be accomplished using centrifugal type separators, and stationary screens. Since these latter units have seen only limited use in the Province, they will not be covered in the following discussion.

8.1 GRIT CHANNELS

Grit channels are usually employed in smaller plants. Grit removal is accomplished by velocity control provided by proportional weirs. Grit channels are normally manually-cleaned. The design parameters for grit channels are as follows:

Number of channels
- at least 2 (with one out-of-service there should be enough capacity in remaining units to handle the peak flow);

Control velocity
- 0.3 m/s;

Control weirs
- proportional, Sutro (or Parshall in parabolic channels);

TABLE 8.1

QUANTITIES OF GRIT REMOVED BY MINISTRY OF THE ENVIRONMENT WATER POLLUTION CONTROL PLANTS (1977)

REGION	QUANTITIES (mL/m^3)	
	RANGE	MEAN
SOUTHWESTERN	1.9-43.6	8.4
WEST-CENTRAL	2.5-22.4	7.6
CENTRAL	1.3-23.0	7.1
SOUTHEASTERN	[illegible] 33.0	3.9
NORTHEASTERN	3.7-27.4	13.3
NORTHWESTERN	1.9-29.0	8.1

<u>NOTE:</u>

1. (mL/m^3) denotes millilitres of grit removed per cubic metre of treated sewage.

Minimum channel width
- 380 mm;

Minimum length
- that required to settle 0.2 mm particle with a specific gravity of 2.65 plus 50 per cent allowance for inlet and outlet turbulence;

Grit storage
- with permanently positioned weirs, the weir crest should be kept 150 to 300 mm above the grit channel invert to provide for storage of settled grit (weir plates that are capable of vertical adjustment are preferred since they can be moved to prevent the sedimentation of organic solids following grit cleaning).

8.2 DETRITUS TANKS

Detritus tanks should be designed with sufficient surface area to remove the same, or smaller, particle size and density as required for grit channels at the expected peak flow rate. Detritus tanks, since they are mechanically-cleaned and do not need dewatering for cleaning, do not require multiple units, unless economically justifiable.

The grit settled in the detritus tank will have a significant organic content due to the lighter solids settling out during low flow periods. Separation of the organics from the grit before, during, or after the removal of the settled contents of the tank can be accomplished in one of the following ways:

- compressed air can be diffused into the tank periodically to re-suspend organic material;

- the removed detritus can be washed in a grit washer with the organic laden wash water being returned to the head of the detritus tank;

- a classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank;

- the removed detritus can be passed through a centrifugal-type separator.

8.3 AERATED GRIT TANKS

Aerated grit tanks for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, should be designed in accordance with the following parameters:

Detention time

- 2 to 5 minutes at peak sewage flow rate (the longer retention times provide additional benefit in the form of pre-aeration);

Air supply

- 4.5 to 9.0 L/m.s (litres per linear metre of tank per second), via wide band diffusion header positioned lengthwise along one wall of tank; (air supply should be variable);

Inlet conditions

- inlet flow should be parallel to induced roll in tank;

Baffling

- minimum of one transverse baffle near outlet weir, with additional transverse baffles in long tanks and longitudinal baffles in wide tanks;

Outlet conditions

- outlet weir oriented parallel to direction of induced roll;

Tank dimensions

- lower limit of above aeration rates generally suitable for tanks up to 3.7 m deep and 4.3 m wide; wider, or deeper tanks require aeration rates in the upper end of the above range; long, narrow aerated grit tanks are generally more efficient than short tanks and produce cleaner grit; L/W ratio normally is 1.5:1 to 2:1, but up to 5:1 is acceptable; Depth/Width ratio 1:1.5 to 1:2;

Desired velocities

- surface velocity in the direction of roll in tanks should be 0.45 to 0.6 m/s (tank floor velocities will be approximately 75 per cent of above);

Grit collectors

- air lifts, pumps, mechanical conveyors or clam shell buckets may be used for the

removal of grit (pre-treatment in the form of screening will be required upstream of mechanical grit removal processes);

Grit washing

- depending upon the method of removal and ultimate disposal, the grit may have to be washed after removal by devices of the type discussed in the previous section;

Multiple units

- generally not required unless economically justifiable, or where grit removal method requires by-passing of tank (as with clam shell bucket);

Tank geometry

- critical with respect to location of air diffusion header, sloping tank bottom, grit hopper and fitting of grit collector mechanism into the tank structure. Consultation with Equipment Suppliers is advisable.

SECTION 9

9.0 PRIMARY SEDIMENTATION

The need for and the design of primary sedimentation tanks will be influenced by various factors, including the following:

- the characteristics of the raw wastewater; the type of sludge digestion system, either available or proposed (aerobic digestion should not be used with raw primary sludges);

- the presence, or absence, of secondary treatment following primary treatment;

- the need for handling of waste activated sludge in the primary settling tank;

- the need for, or possible economic benefits through, phosphorus removal in the primary settling tank(s).

In the past, due to the high capital cost of anaerobic digestion systems, small activated sludge plants tended to avoid their use and instead eliminate primary sedimentation and use aerobic digestion. With the changing energy situation, the potentially lower energy requirements of anaerobic digestion systems make them once more attractive for small activated sludge plants. Furthermore, with the benefits of anaerobic sludge over aerobic sludge in terms of plant-available nitrogen, and therefore its suitability for utilization in farmland, and higher digested sludge solids concentrations, the sludge disposal costs could be expected to be lower with anaerobically digested sludges.

With each new plant, or major expansion o
existing plant, the designer is, therefore
requested to economically compare the waste treatment and sludge treatment alternatives before finalizing the overall process.

Primary sedimentation treatment offers low cost suspended solids and BOD_5 removal, especially in cases where the raw sewage contains a high proportion of settleable solids, as is often the case with sewage containing significant food processing, or similar wastes.

As shown in Table 6.1, primary sedimentation tanks used for phosphorus precipitation with normal strength municipal wastewaters exhibit BOD_5 and suspended solids removals of 65 and 85 per cent, respectively. Without chemical addition for phosphorus removal, the BOD_5 and suspended solids reductions would be 35 and 65 per cent, respectively. With secondary treatment plants, the use of the secondary clarifiers for phosphorus removal has been the most common approach. This has been at least partially due to the reduced chemical requirements when the secondary units are used for phosphorus removal. In view of the potential for increased BOD_5 and suspended solids removals when the primaries are used for phosphorus removal, there may be circumstances when consideration should be given to their use rather than the secondaries for phosphorus removal. Such circumstances might include the following:

- where economic evaluation shows the process to be more cost effective despite the higher chemical costs;

- where existing aeration tanks are overloaded;

- where nitrification is made a requirement for an existing secondary plant;

- where excessive waste activated sludge production is causing anaerobic digester operating problems.

The use of the primaries for phosphorus removal will generally permit removal down to the 1.0 mg/L level, but if lower phosphorus levels are required, chemical addition to the primaries may not be successful. This problem is at least partially due to the fact that some forms of phosphorus are more amenable to precipitation after aeration and that the phosphorus level variations are generally greater in raw sewage than experienced in the aeration tank effluent. It is therefore recommended that precipitation testing be carried out before a final decision is made on which plant treatment units are to be used for phosphorus removal.

Table 9.1 shows the recommended design parameters for primary sedimentation tanks. The recommended surface settling rates should be used for design unless the designer can demonstrate that higher surface settling rates can be tolerated and still achieve the required treatment efficiency. For instance, for plant expansions, it may be possible to show through full-scale testing of the existing primary treatment units that higher surface settling rates will produce the desired results. In such a case, higher surface settling rates will be

considered for approval by the Ministry of the Environment.

Primary sedimentation tanks can be either rectangular or circular. With rectangular tanks, length-to-width ratios of at least 4:1 are preferred. Width-to-depth ratios of 1:1 to 2.25:1 are normal.

In all cases, scum collection and removal facilities should be provided. Scum pits may require heating to avoid freezing problems. Smooth-walled pipe should be used for scum lines to minimize grease buildup. Glass-lined pipe has proven successful for scum piping. Scum lines should be provided with clean-outs or steam injection points to minimize blockage problems due to grease buildup. Scum treatment is normally provided by digestion, but this can lead to problems in the digesters. As an alternate approach, scum may be transferred directly to landfill with screenings or to dewatering or incineration units, if available.

Some pros and cons of rectangular and circular primary sedimentation tanks are as follows:

<u>Rectangular Tanks</u>

- permit common wall construction;
- usually result in a thicker sludge;
- usually less expensive to cover;
- traveling-bridge type collectors may be less expensive than rotary circular collectors for large tanks.

Circular Tanks

- rotary circular sludge collector mechanism usually less costly for small tanks and requires less maintenance than chain and flight-type collectors for rectangular tanks;

- sludge sump can be equipped with a blade to provide stirring to avoid sludge "hang-up";

- usually more susceptible to short-circuiting;

- for tank depths greater than 3 m, may be less expensive than rectangular tanks.

Refer also to SECTION 11 for a discussion of inlet and outlet design.

TABLE 9.1

PRIMARY SEDIMENTATION TANK DESIGN PARAMETERS

PLANT TYPE	TANK DEPTH (m)		SURFACE SETTLING RATE $(L/m^2 \cdot s)$ (1.)		WEIR OVERFLOW RATE $(L/m \cdot s)$ (1.)	
	No P (2.) Removal	With P (2.) Removal	No P (2.) Removal	With P (2.) Removal	No P (2.) Removal	With P (2.) Removal
Primary Treatment Plant	3.0–4.6	3.0–4.6	≤ 0.81	0.41–0.46 (with Alum or Ferric) 0.52 (with Lime)	1.74–5.21	1.74–5.21
Secondary Treatment Plant With W.A.S. Handling In Primaries	3.0–4.6	3.0–4.6	0.58–0.69	--	-- (3.)	-- (3.)
Secondary Treatment Plant Without W.A.S. Handling In Primaries	3.0–4.6	3.0–4.6	0.93–1.39	--	-- (3.)	-- (3.)

NOTES :

1. At peak flow rate.
2. Status of phosphorus removal refers to whether the primary sedimentation tank itself is used for P removal.
3. Weir overflow rates not generally critical provided tank depth is sufficient and weirs are located away from the area of upturn of the sludge density currents.

SECTION 10

10.0 AERATION

10.1 GENERAL

The design of aeration cells and associated equipment will be influenced by various factors including the following:

- expected oxygen demands, including variations, exerted by waste water flows from upstream treatment units;

- hydraulic loading rates, including variability;

- treatment requirements, including necessary reduction of BOD_5, and nitrification;

- temperature, relative rate of oxygen transfer (Alpha factor), and relative oxygen saturation values (Beta factor) for the waste water.

The design parameters for the aeration systems associated with various treatment processes are given in Table 10.1.

10.2 AERATION SYSTEM ALTERNATIVES

Both mechanical and diffused aeration systems should be considered. Refer to the previous section "Component Back-up Requirements" for a discussion of the standby equipment requirements for both types of aeration equipment.

As part of the evaluation of sewage treatment process alternatives, the designer should evaluate the aeration equipment alternatives in terms of the following considerations:

- estimated installed capital cost of equipment;
- expected maintenance costs;
- oxygen transfer efficiencies;
- power requirements;
- estimated operating costs;
- diffuser clogging problems;
- mixing capabilities;
- air pre-treatment requirements;
- aerator tip speed of mechanical aerators used with activated sludge systems;
- icing problems;
- misting problems;
- cooling effects on aeration tank contents.

10.3 OXYGEN TRANSFER EFFICIENCIES AND RATES

The typical oxygen transfer efficiencies and rates, for water at 0 mg/L D.O., 20°C and 101 kPa (1 atm) atmospheric pressure, commonly used for aeration devices are given below:

Coarse bubble diffusers	- 4-6 per cent;
Fine bubble diffusers	- 6-12+ per cent;
Low-speed mechanical aerators (70 rpm, or less)	- 0.42-0.75 kgO_2/MJ or 1.5-2.7 kgO_2/kWh;
Submerged turbine	- 0.28-0.42 kgO_2/MJ or 1.0-1.5 kgO_2/kWh;

High-speed mechanical aerators	- 0.33-0.42 kgO_2/MJ or 1.2-1.5 kgO_2/kWh;
Brush rotors	- 0.42-0.58 kgO_2/MJ or 1.5-2.1 kgO_2/kWh;

Higher oxygen transfer rates than stated above will be considered for approval if the designer can document, or show through pilot-scale testing, that higher rates could be achieved under the expected operating conditions.

There are also other aeration methods, not mentioned above, such as pure oxygen systems, rotating biological contactors, jet aerators, tubular aerators, etc., which will also be considered for approval. Due to the lack of, or only limited experience with these other aeration systems in Ontario, oxygen transfer efficiencies and rates have not been given.

10.4 AERATION BASIN DEPTHS

Aeration basin depth is an important consideration in the design of aeration systems because of the effect that depth has on the aeration efficiency and air pressure requirements of diffused aeration devices and mixing capabilities of mechanical aerators. A minimum aeration basin depth of 3.5 to 4.6 m is recommended for typical sewage treatment plants. Aerated lagoons should have a minimum depth of 3 m. Oxidation ditches should have a minimum depth of 1.6 m.

10.5 MIXING REQUIREMENTS

The aeration system which is selected must not only satisfy the oxygen requirements of the mixed liquor, but must also provide sufficient mixing to ensure that the mixed liquor remains in suspension. One exception to this is with the aerated lagoons of the aerobic-facultative type where mixing is only provided to the extent necessary to ensure uniform D.O. levels in the upper layers of the aeration cell. The power levels necessary to achieve uniform dissolved oxygen and mixed liquor suspended solids concentrations are shown in Table 10.2.

10.6 VARIABLE OXYGENATION CAPACITY

Consideration should be given to reducing power requirements of aeration systems by varying oxygenation capacity to match oxygen demands within the system. Such a system would utilize automatic D.O. probes in each aeration basin to measure dissolved oxygen levels. An output signal could then be used to change the number of aerators in operation, aerator speed, immersion of surface aerator impellers, or air flow to submerged turbine or diffused aeration systems to maintain the required minimum dissolved oxygen levels. Power savings of up to 25 per cent of the normal aeration requirements have been reported based upon European experience with such systems.

Designers are cautioned to check mixing requirements to ensure adequate suspension of MLSS.

TABLE 10.1

AERATION SYSTEM DESIGN PARAMETERS

TREATMENT PROCESS	ORGANIC LOADING RATE ($kg\ BOD_5 / m^3 \cdot d$)	F / Mv (d^{-1}) (1.)	MINIMUM DETANTION TIME (h, Based on Q Avg.)	RETURN SLUDGE RATE (% Q Avg.) (2.)	OXYGEN DEMAND IN TYPICAL MUNICIPAL SEWAGE AT STANDARD CONDITIONS (10.) ($kg\ O_2 / kg\ BOD_5$ Applied or Nitrogen Applied Loading)	SOLIDS RETENTION TIME (SRT) (Days)	MINIMUM DISSOLVED OXYGEN LEVEL (mg/L)	MINIMUM RESIDUAL ALKALINITY (mg/L as $CaCO_3$)
CONVENTIONAL A. S. (3.)								
- WITHOUT NITRIFICATION	0.31-0.72	0.2-0.5	6	25-100	1.0 kg/kg BOD_5	4-8	2.0	-
- WITH NITRIFICATION (4.)	0.31-0.72	0.05-0.25	6	25-100	1.0 kg/kg BOD_5 + 4.6 kg/kg NH_4 - N	>4 at 20°C; >10 at 5°C	2.0	50
EXTENDED AERATION								
- WITHOUT NITRIFICATION	0.17-0.24	0.05-0.15	15	50-200	1.5 kg/kg BOD_5	>15	2.0	-
- WITH NITRIFICATION	0.17-0.24	0.05-0.15	15 (11.)	50-200	1.5 kg/kg BOD_5 + 4.6 kg/kg TKN	>15	2.0	50
HI-RATE								
- WITHOUT NITRIFICATION	0.72-0.96	0.3-0.5	4	50-200	1.0 kg/kg BOD_5	4-6	2.0	-
- WITH NITRIFICATION (5.)	-	-	-	-	-	-	-	-
CONTACT STABILIZATION								
- WITHOUT NITRIFICATION	0.31-0.72 (6.)	0.2-0.5 (6.)	0.33 (7.); 4 (8.)	50-150	1.0 kg/kg BOD_5	4-10	2.0	-
- WITH NITRIFICATION (5.)	-	-	-	-	-	-	-	-
AERATION LAGOONS (9.)								
- WITHOUT NITRIFICATION	0.031-0.048	-	-	-	1.0 kg/kg BOD_5	-	2.0	-

NOTES:

1. "F" is the mass loading to the aeration tank of BOD_5 per day and "Mv" is the mixed liquor volatile suspended solids mass under aeration.
2. Return sludge pumpage should be variable over the full range given.
3. Including step aeration.
4. Refer to "Nitrification in Activated Sludge Plants, Guidelines on Some Operation and Design Aspects", research publication W62 Revised July, 1977.
5. Hi-rate and contact stabilization not considered suitable for nitrification.
6. Considering contact and re-aeration tankage.
7. Based on Q Peak + 100% Q Avg return sludge rate, (Contact).
8. Based on 100% Q Avg return sludge rate, (Re-aeration).
9. Aerobic-Facultative Lagoons (without separate clarification and without sludge return) providing pre-treatment prior to conventional lagoons.
10. The designer must adjust these values to the necessary O_2 transfer rate of the chosen aeration equipment by applying factors for alpha, beta, D.O. and non-standard conditions such as altitude and temperature.
11. If nitrification is required year round, a longer detention time may be required.
12. Deviations from the recommended design parameters may be permitted if the designer can demonstrate through operating data or test that such deviations can be tolerated and still achieve the required treatment efficiency.

TABLE 10.2

AERATION MIXING REQUIREMENTS

(1.)

AERATION SYSTEM	FOR UNIFORM D.O. LEVELS	FOR UNIFORM MLSS LEVELS
Mechanical	1.6 to 2.5 W/m^3	16 to 25 W/m^3
Diffused (Coarse Bubble, Spiral Roll)	-	0.33 $L / m^3 \cdot s$ (2.)
Diffused (Fine Bubble Domes, Full Floor Coverage)	-	0.61 $L / m^2 \cdot s$ (3.)

NOTES :

1. Mixing requirements vary with basin geometry, MLSS concentrations, placement of aeration devices, pumping efficiency of aerators, etc. Wherever possible, refers to full-scale testing results for the particular aerator being considered.
2. $L / m^3 \cdot s$ refers to volume of air per second per volume of aeration tank.
3. $L / m^2 \cdot s$ refers to volume of air per second per horizontal cross-sectional area of aeration tank.

10.7 FINE BUBBLE DIFFUSER SYSTEMS

With increased emphasis being placed on energy conservation in sewage treatment plant design, fine bubble diffuser systems are coming into more common use in Ontario. Such systems have oxygenation efficiencies of approximately 12 per cent with conventional tank depth (4.6 m) and greater efficiencies with increased depths.

Due to their increased efficiencies, such systems utilize lower air flows and therefore have lower mixing capabilities than conventional diffusers. As a result, precautions must be taken to ensure that proper mixing will occur. In addition, these fine bubble systems may under certain circumstances foul with slime. Although the slime formation is believed to be caused by a number of factors including high F/M ratios, high soluble BOD_5 levels, and low dissolved oxygen concentrations, low mixing levels will also exacerbate sliming [66].

Some of the points which should be considered in the design of fine bubble diffuser systems are as follows:

a) pilot testing should be carried out wherever possible to determine if sliming will occur. This is particularly important where industrial waste contribution is significant or where soluble BOD_5 concentrations are expected to be high due to other causes;

b) manufacturer's recommended maximum and minimum air flow rates should be complied with; for 178 mm diameter domes, minimum and maximum air flow rates should be 0.233 and 0.933 L/s;

c) the ability to vary air flow rates should be possible to take full advantage of the efficiency of the diffusers and to minimize fouling; when satisfying peak O_2 demands diffusers should be operating at close to their maximum air flow rating; air valves should be provided for each grid of the aeration system; automatic variation of air flow rate is desirable, but as a minimum a D.O. probe should be located in the area of the aeration tank near the raw sewage inlet and set to alarm if D.O. falls to 1 mg/L;

d) to facilitate dome cleaning, equipment should be provided to allow for rapid tank draining, diffuser removal and diffuser cleaning, cleaning of ceramic domes may be carried out by hosing and scrubbing, steaming, firing or acid cleaning, or combinations of these methods; a clean water source should be available to refill the tanks following cleaning;

e) very efficient air cleaning must be provided; replaceable air filters, using coarse pre-filters and fine final units, may be the simplest and least expensive; electrostatic precipitators or bag houses may also be used; with retrofit plants old

piping may have to be replaced since the flaking of rust from cast iron lines may clog the diffusers from inside; equipment should be provided to remove liquid accumulations from inside the headers following power failure or repair shut downs;

f) spare parts should be provided including diffusers, gaskets, bolts and air supply piping;

g) since dome diffusers are better vertical mixers than horizontal mixers, tanks should be built as deep as possible to minimize the horizontal travel required; oxygen transfer efficiency, however, may taper off at depths greater than 6.1 m due to oxygen depletion in the air bubbles;

h) although diffusers work best under conditions of uniform loading, some degree of plug flow appears to be desirable for good sludge settleability; length (L) to width (W) ratios of approximately 8:1 (where L=8W) are recommended; a sludge reaeration zone area of W x W at the tank inlet end, with sewage inlet downstream of this zone is recommended (return sludge channel aeration can be used instead); beyond the sludge reaeration zone, the aeration system should be divided into approximately 4 grids with the number of dome diffusers per grid being gradually reduced to match O_2 supply to demand; full-floor coverage should be provided in the first 3/4 of the aeration tank; in the last 1/4 of the aeration tank, the dome diffusers should be

positioned along the centreline of the tank to induce a double spiral roll mixing effect; to avoid over-design of the O_2 supply, 50% blanks should be provided in at least the first half of the aeration system for possible addition of more diffusers if found necessary; step feeding to at least mid-tank length should also be allowed for in design in case it is needed to reduce sliming problems;

i) manufacturers' mixing power recommendations should be adhered to; power levels as low as 10 W/m^3 (wire power level) have been reported as being sufficient to suspend biological solids.

SECTION 11

11.0 SECONDARY SEDIMENTATION

Secondary settling tanks must be designed for the greater of the surface areas required for either clarification or solids handling.

The area requirements for clarification vary with the settling characteristics of the mixed liquor. Factors which can influence the settling characteristics are chemical addition to the mixed liquor for phosphorus removal and the nitrification process.

Table 11.1 gives the recommended design parameters for secondary sedimentation basins.

Scum removal facilities must be provided for secondary clarifiers in all cases. Where freezing would cause equipment damage, provision should be made for removal of scum collectors in winter.

Circular, rectangular, or square clarifiers may be used. In selecting the clarifier shape, the designer should consider the following factors:

- effective use of the site;

- means of future expansion;

- economics of tank construction, including inlet and outlet piping, sludge and scum removal equipment, etc;

- head loss through the system.

TABLE 11.1

SECONDARY SEDIMENTATION BASINS DESIGN PARAMETERS

MIXED LIQUOR TYPE	TANK DEPTH (m)	SURFACE SETTLING RATE ($L/m^2 \cdot s$) (At Peak Overflow Rate)	WEIR LOADING RATE ($L/m \cdot s$)	SOLIDS LOADING ($kg/m^2 \cdot d$) (At Peak Flow)
Activated Sludge (No Chemical Addition to ML for P Removal)	3.6–4.6	0.58	2.9	≤240 (Including 50% Return Sludge)
Activated Sludge (With Chemical Addition to ML for P Removal)	3.6–4.6	0.41	2.9	≤240 (Including 50% Return Sludge)
Activated Sludge (With Nitrification Requirement)	3.6–4.6	0.34	2.9	≤120 (Including 100% Return Sludge)
Extended Aeration (With or Without P Removal)	3.6–4.6	0.41	2.9	≤120 (Including 100% Return Sludge)

NOTES:

1. Weir loading rates for weirs located away from the upturn of the density current should not exceed 2.9 $L/m \cdot s$, and for weirs located within the upturn zone, the rate should not exceed 2.2 $L/m \cdot s$.
2. Peak overflow rate is the peak flow rate entering the sedimentation basin excluding the return sludge flow rate.

Rectangular tanks should satisfy the following geometrical ratios:

> length:width of 4:1, or greater;
> width:depth of 1:1 to 2.25:1.

The inlet to sedimentation basins must be designed in such a way that the flow is distributed across the full cross-sectional flow- through area, while minimizing short circuiting and turbulence factors. Inlet velocities should be kept as low as possible.

With circular basins having 100 per cent sludge recirculation, the inlet well should not be less than 20 per cent of the tank diameter and have a depth of 55 to 65 per cent of the SWD. The maximum flow velocity to the centre inlet well should not exceed 1.0 m/s and the outflow velocity should not exceed 0.08 m/s. Peripheral and spiral feed circular sedimentation tanks are also available, but to-date there has been insufficient experience in Ontario with such systems to allow design parameters to be suggested.

With rectangular tanks, baffled inlet ports are generally used to achieve uniform flow distribution. Maximum inlet port velocities should be in the range of 0.08 to 0.16 m/s.

Outlet weirs must be provided with sufficient effective length and in locations such that the clarified effluent can be withdrawn from the tank without causing excessive localized upflow and resulting solids carryover. Recommended maximum weir loading rates are given in

Table 11.1 For conventional circular tanks, a peripheral weir is generally all that is required to provide a suitable weir rate. With rectangular tanks, multiple weirs will generally be required and these should be located away from the area of upturn of the density current.

Activated sludge should be removed from final sedimentation tanks as quickly as possible to avoid adverse effects on the sludge quality caused by anaerobic conditions. Sludge solids should not remain in the sedimentation basin for more than 30 minutes. Sludge scraper systems may consist of chain-and-flight type or traveling-bridge type for rectangular sedimentation tanks. With circular tanks, rotary circular scraper mechanisms are used. Both traveling-bridge and rotary mechanisms can be of the rapid sludge removal type if they are equipped with suction sludge drawoff pipes. The use of such systems should be considered particularly with large tanks where long sludge residence times would otherwise be experienced.

When sludge is scraped towards a hopper for removal in rectangular tanks, the hopper has normally been located at the inlet end of the tank. Other designs placing the hopper at tank mid-point, or at the effluent end, to take advantage of the density current, have also been used successfully.

Wherever possible, pipes discharging return sludge or waste activated sludge should be located to permit visual confirmation that sludge is being discharged.

SECTION 12

12.0 EFFLUENT DISINFECTION

A draft MOE policy and its draft guidelines cover disinfection requirements for sewage treatment plants. MOE Regional Staff should therefore be consulted to determine the disinfection requirements for any sewage treatment works proposal.

Although some alternative methods of disinfection have been under investigation during recent years, including ultraviolet irradiation, gamma irradiation, ozonation, etc., chlorination is the predominant form of effluent disinfection in full-scale use in Ontario at the present time with the exception of one ultraviolet irradiation system which has had a good performance report and should be evaluated versus other modes of disinfection. This section will, therefore, only cover the currently accepted practice with respect to chlorination.

12.1 CHLORINATION

Although some very small sewage treatment works in Ontario use liquid sodium hypochlorite for chlorination, virtually all the larger plants use liquid-gas chlorine. With changing costs for the products and with the safety concerns with delivery and use of gaseous chlorine, the use of sodium hypochlorite is likely to increase. Designers should therefore consider both chemicals when evaluating chlorination alternatives.

A chlorine residual of 0.5 mg/L is generally required. The required detention time is generally 30 minutes at average dry weather

flow. Where other uses are made of the receiving water which would be adversely affected by occasionally high bacteria levels, higher detention times will be required and consideration given to the peak flow rates requiring disinfection. Under these latter circumstances, the minimum detention time would be 15 minutes at the peak sewage flow rate.

In calculating the detention time, the residence time in the outfall sewer may be taken into consideration. If the outfall sewer is able to provide the full contact time, provision must at least be made for adequate mixing of the chlorine and effluent prior to entering the outfall and facilities provided so that chlorinated effluent samples can be obtained. These requirements can normally be satisfied by constructing a short retention mixing chamber immediately upstream of the outfall sewer.

The disinfection effectiveness of chlorine can be greatly enhanced by thorough mixing of the concentrated chlorine solution with the wastewater effluent prior to the contact tank. In order that the chlorine contact tank can provide the required detention, dead zones within the tank must be avoided and the flow through the tank must approach plug flow as closely as possible. Back-mixing within the contact tank must be avoided to prevent short-circuiting and the resulting poor disinfection results.

To approach a plug-flow regime, flow channels with length-to-width ratios of greater than 40:1 are required. Length-to-width ratios of 10:1

produce detention times of approximately 70 per cent of theoretical residence times. In rectangular tanks, longitudinal baffling to produce long, narrow flow channels with a serpentine flow pattern and with guide vanes at changes in direction, appears to be the best method to produce an efficient contact basin.

Since some sedimentation occurs in chlorine contact tanks, provision should be made for periodic sludge removal from the chlorine contact tank(s). If it is necessary to take the contact tank out of operation for cleaning, and if short-term discontinuation of disinfection cannot be tolerated due to other critical uses made of the receiving waters, two contact basins should be provided. In less critical situations, one contact basin will suffice provided that the by-pass facilities are equipped with a chlorine application point for emergency disinfection.

In order that effective disinfection can be maintained at all times, without the need to overdose excessively at low flow periods, the chlorine feed equipment should be paced by the effluent flow rate. In locations where excessive chlorine residuals would have an adverse effect on the receiving stream, a more elaborate system of pacing, using compound loop control may be necessary, or in some cases, dechlorination may be required.

12.2 OTHER USES OF CHLORINE

In designing the chlorination system, chlorine application should be considered for points other than the chlorine contact tank, as follows:

- influent sewer (for odour control);
- return sludge (for bulking control);
- by-pass sewers (for emergency disinfection);
- upstream of polishing filter (for control of growths in filter beds);
- sludge thickeners (for odour control and maintaining sludge in fresh condition).

12.3 CHLORINE REQUIREMENTS

The ranges of chlorine dosages required for disinfection of various qualities of effluent and raw sewage are as follows:

	Dosage (mg/L)
Raw sewage	6-12 (fresh)
Raw sewage	12-25 (septic)
Primary effluent	3-20
Secondary effluent	2-9
Tertiary filter effluent	1-6

12.4 WATER REQUIREMENTS

Chlorine injector systems require large volumes of water which typically amount to approximately 330 L of water per kg of chlorine used. Higher water requirements can be experienced depending upon the back pressure at the point of injection. To minimize operating costs, effluent water should be used in the injection systems whenever possible, with municipal water, or water from an on-site system, providing the necessary standby supply.

12.5 CHLORINE SAFETY REQUIREMENTS

Liquid-gas chlorine storage and scale rooms and chlorination equipment should be designed in compliance with the Ministry of Labour, Industrial Safety Branch, Engineering Data Sheet No. 5-3 "Storage and Use of Chlorine". Where applicable, the Ministry of the Environment's Technical Bulletin 65-W-4 "Chlorination of Potable Water Supplies" may also be used for guidance with respect to equipment, building details, etc. Both of these documents are included in Appendix E.

For more information on chlorination systems, refer to Section 15.0 and references, such as [19] and [28].

SECTION 13

13.0 EFFLUENT FILTRATION

Effluent filtration is generally necessary when effluent quality better than 15 mg/L BOD_5 and suspended solids is required. With proper pre-treatment and conservative filtration system design, effluent quality as low as 5 mg/L BOD_5 and suspended solids and 0.3 mg/L total phosphorus can be achieved.

There are various types of effluent filtration systems which are available including: single, dual and mixed-media systems; shallow and deep bed systems; upflow and downflow filters; gravity and pressure systems; continuous and discontinuous operation filters; manual and automatic- backwash filters, slow sand filters, etc. Due to the significant differences between the various pieces of equipment, it is impossible to fully cover all parameters in these guidelines. Some general design preferences for such systems are listed below:

a) Effluent filtration should precede the chlorine contact chamber to minimize chlorine usage, to allow more effective disinfection and to minimize the production of chloro-organic compounds.

b) To allow excessive biological growths and grease accumulations to be periodically removed from the filter media, a chlorine application point should be provided upstream of the filtration system (chlorine would only be dosed as necessary at this location).

c) Filtration rates at peak sewage flow rates, including backwash flows, should not exceed 2.1 $L/m^2.s$ for shallow bed single media systems (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid under-sizing of the filter. See note below).

d) Filtration rates at peak sewage flow rates, including backwash flows, should not exceed 3.3 $L/m^2.s$ for deep bed filters (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid undersizing of the filter. See note below). The manufacturer's recommended maximum filtration rate should, however, not be exceeded.

e) Peak solids loading rate should not exceed 51 $mg/m^2.s$ for shallow bed filters and 83 $mg/m^2.s$ for deep bed filters (if raw sewage flow equalization is provided, lower peak solids loading rates should be used in order to avoid undersizing of the filter. See note below).

NOTE: No firm recommendations can be made for lower peak filtration/solids loading rates because of the high number of alternatives to be considered. The intention is to take advantage of the flow equalization facility in determining the filter area but not to the extent which would result in an undersized filter. Rationalization of the selected filtration/solids loading rates should be made available to the design approving authority by the designer.

f) Backwash rates should be at least $10\ L/m^2.s$, or whatever rate is necessary to achieve at least 10 per cent bed expansion.

g) Air scour or mechanical agitation systems to improve backwash effectiveness are recommended.

h) If instantaneous backwash rates represent more than 20 per cent of the average daily design flow rate of the plant, a backwash holding tank should be provided to equalize the flow of backwash water to the plant.

i) Backwash waters should be returned to the primary sedimentation tanks or to the effluent end of the aeration tanks (if there is no primary sedimentation section).

Factors to consider when choosing between the different filtration systems which are available, include the following:

a) the installed capital and expected operating and maintenance costs;
b) the energy requirements of the systems (head requirements);
c) the media types and sizes and expected solids capacities and treatment efficiencies of the systems;
d) the backwashing systems, including type, backwash rate, backwash volume, effect on sewage works, etc.

For more information on effluent filtration systems, references [29] and [30] should be consulted.

SECTION 14

14.0 PHOSPHORUS REMOVAL

Phosphorus removal requirements for sewage treatment plants are covered in a draft MOE policy and its draft guidelines. MOE Regional staff should be consulted regarding the particular requirements for any sewage treatment plant proposal.

With phosphorus removal having been required at many Ontario plants since December 1973, there is a great deal of data available to assist designers in engineering systems for new water pollution control plants. References [31], [32], [33], [34], [35], [36], [37] and [38] should be consulted for additional design information and Section 15.0 referred to for chemical handling recommendations.

For the typical chemical dosage requirements of various chemicals and sewage treatment processes, refer to Table 14.1.

Due to problems which have been experienced with the handling of lime, the vast majority of treatment plants now use either ferric chloride or alum for phosphorus removal. For new treatment plants, these latter two chemicals are likely to be the most suitable chemicals to use and, wherever possible, the facilities should be designed to permit either of these two chemicals to be utilized.

In designing the chemical feed system for phosphorus removal, the following points should be considered:

a) need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemical(s);
b) selection of chemical feed equipment with the required range in capacity;
c) need for standy-by chemical feed pump;
d) provision of flow pacing for chemical pumps proportional to sewage flow rates;
e) flexibility by providing a number of chemical application points;
f) need for protection of storage and piping from the effect of low temperatures;
g) selection of the proper chemical storage volume;
h) need for ventilation in chemical handling rooms;
i) provision for containment of any chemical spills.

Total phosphorus reduction to the 1 mg/L level can generally be accomplished by chemical coagulation and sedimentation in settling tanks designed in accordance with the parameters outlined in Tables 9.1 and 11.1. If phosphorus removal to the 0.3 mg/L level is required for mechanical plants, effluent filtration will generally be necessary in addition to chemical precipitation. Batch chemical dosage of seasonal retention lagoon systems may be able to achieve 0.3 mg/L effluent phosphorus levels.

With secondary treatment plants, the chemical dosage requirements for either alum or ferric chloride have been found to be least when the addition of chemical is made to the aeration tank effluent. Dosage to the aeration tank influent requires as much as 35 per cent higher dosage rates.

TABLE 14.1

PERMANENT INSTALLATION CHEMICAL DOSAGE REQUIREMENTS FOR PHOSPHORUS REMOVAL

TYPE OF TREATMENT PLANT	ADDITION POINT	DOSAGE RATES (mg/L) (2.)			
		CHEMICAL	RANGE	AVERAGE	NO. OF PLANTS
PRIMARY (1.)	Raw Sewage	Alum	100	100	1
		Ferric Chloride	6-30	16	9
		Lime	167-200	185	3
SECONDARY (1.)	Raw Sewage	Lime	40-100	70	2
		Alum	-	-	0
		Ferric Chloride	-	-	0
	Secondary Section	Lime	-	-	0
		Alum	30-150	65	15
		Ferric Chloride	2-30	11	32
WASTE STABILIZATION PONDS (3.)					
Seasonal Retention Lagoons	Batch Dosage To Cells	Alum	100-210	163	5
		Ferric Chloride	17-22	20	4
		Lime	250-350	300	1
Continuous Discharge Lagoons	Raw Sewage	Alum	225	225	1
		Ferric Chloride	20	20	1
		Lime	400	400	1

NOTES :

1. From Canada-Ontario Agreement Report No.83
2. Dosage Values for Alum as $Al_2(SO_4)_3 \cdot 14\ H_2O$
   Ferric Chloride and Ferric Sulphate as Fe^{+++}
   Hydrate Lime as $Ca(OH)_2$
3. From Canada-Ontario Agreement Reports No. 13 and 65.

SECTION 15

15.0 HANDLING OF CHEMICALS

15.1 GENERAL

Chemicals selected for use in sewage treatment plants must be such that they will not adversely affect the operation of the sewage or sludge treatment processes and will not leave dangerous residuals in the effluent or sludge leaving the plant. The purity of chemicals proposed to be used should be determined. Occasionally, waste streams from industry, such as ferrous chemicals, can be used provided that they are not contaminated with other hazardous materials.

The general types of chemicals required will depend on the process requirements, and the specific chemicals selected for treatment will depend on the economic considerations; for example, when disinfection is required it may be provided either by sodium hypochlorite, or chlorine gas, and where phosphorus removal is required, it may be provided by alum or ferric chloride compounds.

The designer is referred to references [40], [41] and [42] for general guidance on chemical selection and handling, and should obtain further information from chemical suppliers to ensure optimum design.

15.2 STORAGE OF CHEMICALS

Some of the considerations which will have an effect on the chemical storage volume requirements are as follows:

- chemical usage rate and delivery time (usually at least 7 days storage should be provided);

- typical volume delivered (storage should be at least one truck load plus 25 per cent);

- availability of alternate suppliers;

- loss in strength of chemical with storage time;

- seasonal changes in chemical strength;

- critical nature of chemical with respect to treatment process.

Structures, rooms, and areas accommodating chemical storage and feed equipment should be arranged to provide convenient access for chemical deliveries, equipment servicing and repair, and observation of operation. It is recommended that wherever possible the storage area be separated from the main plant, and that segregated storage be provided for each chemical. Where two, or more, chemicals could react with undesirable effects, the drainage piping (if provided) from the separate chemical handling areas should not be interconnected. For dangerous materials such as gaseous chlorine, either floor drains in the storage and scale rooms should be omitted entirely, with the floors sloped towards the doors, or floor drains installed, but kept totally separated from the drainage systems for the rest of the building.

It is strongly recommended that all chemical storage be at or above the surrounding grade. Where subsurface locations for chemical storage tanks are proposed, these locations shall be free from sources of possible contamination, and positive drainage for ground waters, chemical spills, and overflows must be assured. Where above grade storage is provided, due consideration should be given to the method of unloading chemicals; for example, there is a limit on the allowable pressures to be used for air-padded trucks. Where dry chemicals are used, it is recommended that some form of loading dock or ramp be provided.

The storage areas and locations should be arranged to prevent any chemical spills, and to facilitate clean-up operations, the floor surfaces should be smooth, impervious, slip-proof, and adequately sloped to drainage points.

Adequate measures should be taken to provide a minimum temperature of 15°C for chlorine gas areas, and the remainder of the chemical buildings should be heated to a temperature to prevent crystallization or freezing of chemical, or abnormally high viscosities (polyelectrolytes) which would make pumping difficult.

The ventilation system for chemical buildings should be such that exhausted air is passed outside the building, and arranged within the building to provide for slight negative pressures in areas where dry chemicals are in use, as a dust control measure. Where large amounts of dust are anticipated from chemical handling operations, adequate air filtration equipment should be included in this ventilation system.

All chemical buildings should be provided with eye wash units and/or deluge showers, adequate facilities for cleaning up chemical spills, space for cleaning and storage of the recommended protective equipment, and adequate warning signs, conspicuously displayed where identifiable hazards exist. It is recommended that all doors in chemical buildings open outward, and that corridors or space between storage areas be a minimum 1.5 m wide to permit the use of hand trucks, etc.

15.2.1 Liquid Chemicals

All storage tanks shall be constructed of a material proven for the intended service. Since some chemicals, such as ferric chloride, are delivered at very high temperatures (up to 60°C), the tanks and associated equipment should be able to withstand such temperatures. Tanks located outside should be heat traced and insulated to maintain the minimum allowable temperature for the chemical being stored.

All storage tanks shall be provided with an adequate size fill line, minimum 50 mm diameter, which is sloped to drain into the tank. The fill line should be adequately identified at the end remote from the tank, and provision should be made to drain this fill line, if a "down leg" exists.

Each tank should have an adequate vent line, minimum size 50 mm, with a down turned end. Where venting outside the room is required, the vent should be provided with an insect screen.

All tanks shall have an overflow adequate for the rate of fill proposed for the tank, and overflows shall be sloped down from the tanks, with ends turned down and having insect screens, and shall have a readily visible free discharge directed to a suitable containment area. Tanks to be filled by pumping should be provided with an overflow not less than 300 mm above the design level, and not less than 150 mm above the design level when filling is done by gravity flow.

Each tank should be provided with means to indicate the level of contents in the tank, and where an external level gauge is provided, it shall have a shut-off valve at the tank connection. Each tank should be equipped with a drain. Tanks should have removable lids or covers or manholes where the contents are such that venting indoors is permitted. In the case of tanks which are to be vented outside, the covers or manholes should be constructed so as to be air tight.

Tanks should be arranged to provide a minimum clear space all around of not less than 300 mm. Where tanks with liners are used, weep holes shall be provided in the outer shell to show positive indication of liner leakage.

A containment system should surround liquid storage tanks to contain spills.

All storage tanks should be conspicuously signed with the contents and principal hazards of the tank shown.

15.2.2 Dry Chemicals

Where dry chemicals are to be used, provision must be made to minimize dust problems in handling. The use of granular materials is preferred.

Particular care should be taken with fine dusts around electrical equipment, and where exhaust fans, filters, and vacuum conveying systems are used, grounding should be provided to prevent any static electricity build-up.

15.2.3 Liquid-Gas Chlorine

Refer to Appendix E for the general requirements for the storage of chlorine.

As previously indicated, floor drains from chlorine storage or scale rooms should not be connected with drainage systems of other parts of the building, or other buildings. Chlorine gas is heavier than air and could travel via floor drains, foundation drains, etc. into other rooms. If floor drains are to be used, they should be completely separated from other drainage systems. As an alternative to floor drains, the floors may be sloped towards doors to provide the needed drainage.

Gas detectors and alarms should be provided for storage and scale rooms. If the plant is not continuously manned, the alarm should terminate at a fire station, police station or other 24-hour manned location.

Each sewage treatment works using liquid-gas chlorine should have a contingency plan to deal with major gas leaks.

Chlorine gas feed and storage rooms should be provided with inspection windows to permit viewing of the interior of the room and equipment. Switches for fans and light should be outside the room at the entrance, and a signal light indicating fan operation should be provided at each entrance. Vents from feeders and storage shall discharge to the outside atmosphere above grade, and should slope down wherever possible.

15.3 CHEMICAL APPLICATION POINTS

All chemicals should be applied to the waste streams or sludges at such points, and in such a way as to ensure the maximum efficiency for treatment, and to provide maximum safety to the operators. Chemicals should be added at a point of turbulence, or through a diffuser to ensure satisfactory mixing. Particular care should be taken where the point of addition is close to a point where flows split. Alternate chemical addition points should be provided to give maximum flexibility of operation where appropriate. Where chemicals are added to lines under pressure, a suitable isolating valve should be provided.

With phosphorus removal chemicals, it is desirable to terminate the chemical feed point above the liquid level of the tank or channel so that the chemical discharge can be observed by the operator.

15.4 CHEMICAL FEED EQUIPMENT

Where the chemical added is necessary for the protection of the receiving waters such as chlorination, phosphorus removal, or other critical processes, a minimum of two feeders shall be provided, with one acting as a stand-by unit.

The design and capacity of feeders should be such that they will be able to supply at all times, the necessary amounts of chemicals at an accurate rate, throughout the feed range. Feeders shall be capable of proportioning the chemical feed to the rate of flow.

Chemical solutions can be prevented from being siphoned into the waste stream by either assuring discharge at a point of positive pressure or providing vacuum relief or a suitable air gap.

All positive displacement pumps should be equipped with adequately sized pressure relief valves. If the pumped fluid is relieved through this valve, it must pass to a safe location, preferably back to the storage tank. Where liquid filled diaphragm pumps are in use, the over pressure should be relieved internally or by discharge of the motive fluid to a safe location. Pressure relief valves should be set at a safe relief pressure to avoid damage to the chemical feed lines.

Chemical feed lines should be kept as short as possible, protected against freezing, and readily accessible through their entire length.

The minimum line size shall be 12 mm, unless the material pumped exhibits scale forming tendencies, then the minimum size shall be 25 mm. In general, the feed line should be designed to be consistent with the scale forming or solids depositing properties of the material conveyed. Where feed lines are provided from duplicate pumping units, or passed to a distribution manifold, adequate valving shall be provided to isolate sections of the line.

Where reciprocating type pumps are to be used, it is recommended that flexible connections be provided on the pump suction and discharge to prevent the transmission of vibrations to the feed line. These flexible connections should be sufficiently rigid to withstand both the pump suction and discharge pressure, and reinforced hose is recommended. The pump, in combination with its suction piping and valving arrangement, shall be such that the pump discharge rate remains the same regardless of the level of chemical in the feed tank.

Where dry chemical feeders and storage equipment are provided, the design of the storage equipment shall be adequate to prevent bridging or other problems in the storage silo. Dry chemical feeders will be acceptable if they measure either volumetrically or gravimetrically, and provide for effective solution of chemical in a solution pot. The dry chemical feed system should be enclosed to prevent emission of chemical dusts into the operating room.

Feeders may either be manually or automatically controlled, and automatic control must revert to manual control as necessary. Feed rates proportional to flows must be provided.

Where solution tanks are to be used, means should be provided to maintain a uniform strength solution, and continuous agitation should be provided to maintain slurries in suspension. Normally two solution tanks will be required to ensure continuity of supply in servicing the solution tank. Each tank should be provided with a drain.

Make-up water for the solution tank shall enter the tank not less than 150 mm above, or two pipe diameters, whichever is greater, above the maximum solution level unless the make-up water supply is protected with an approved back flow prevention device.

Where the design of the chemical feed system includes day tanks, such day tanks should have a maximum capacity equivalent to the chemical consumed over a 30 h period. Day tanks should either be scale mounted or have a calibrated level gauge provided. The piping arrangement for refilling the day tanks should be such that it will prevent over filling of the tank. In all other respects, the requirements for day tanks should conform with the requirements for bulk storage tanks.

SECTION 16

16.0 SLUDGE DIGESTION

A draft MOE policy and its draft guidelines cover the stabilization and/or dewatering requirements for the various sludge utilization/disposal options. MOE Regional staff should therefore be consulted to determine the stabilization and/or dewatering requirements for any planned sludge utilization/disposal operation.

Since sludge stabilization is generally achieved by digestion, these guidelines will not deal with other sludge stabilization methods. Two types of sludge digestion systems are in common use in Ontario - anaerobic (mesophilic) and aerobic.

Anaerobic digestion is the most commonly used system for the digestion of primary and mixtures of primary and waste activated sludges. Aerobic digestion, on the other hand, has normally been used in Ontario for the stabilization of waste activated sludges from water pollution control plants which do not have primary settling tanks. Although there are some aerobic systems treating mixtures of raw primary and waste activated sludges, due to the higher oxygen requirements and, therefore, higher energy costs to aerobically digest raw primary sludges, it is recommended that the process generally not be used for such sludges in new plants.

In certain plants, separate handling of the waste activated sludge in aerobic digesters and primary sludge in anaerobic digesters may be economically justifiable.

Designers are cautioned to give thorough consideration to not only what type of digestion will best suit a particular treatment plant, but what type of overall system, including plant type and digestion type will produce the desired results at least cost. Sludge handling, digestion and disposal are becoming such a large portion of the overall sewage treatment operation, that no one unit operation can be considered in isolation from the other plant components.

In the following sub-sections, the suggested design guidelines for anaerobic and aerobic digestion systems are given. For the Ministry's guidelines on sludge spreading on agricultural land[1]; reference should be made to "Guidelines for Sewage Sludge Utilization on Agricultural Lands" [64].

16.1 SLUDGE QUANTITIES AND CHARACTERISTICS

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data

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1. For sludges intended to be utilized on agricultural land, anaerobic or aerobic digestion must be provided. Extended aeration plant waste activated sludges treated in aerated holding tanks will only be suitable for utilization on farmland if the aerated holding tank provides the equivalent treatment of aerobic digestion, in terms of sludge age and aeration rate.

from existing plants may be unsuitable for use in design. Before sludge data is used for design, it should be assessed for its accuracy. When reliable data are not available, the sludge generation rates and characteristics given in Table 16.1 may be used.

16.2 ANAEROBIC DIGESTION

Anaerobic digestion systems produce digester gas which has, as its main constituent, methane. To safeguard anaerobic digester and gas handling system design, CAN1-B105-M81 "Installation Code for Digester Gas Systems" [43] has been prepared.

At present, the Canadian Gas Association, if requested, will carry out a review of designs of anaerobic digester gas systems on a fee for service basis prior to construction. Certification of the gas systems will only be granted following inspection of the constructed works by the Canadian Gas Association.

Digestion systems should be designed with features and in accordance with design parameters, as follows:

Number of stages

- two;

Number of digesters in each stage

- in small plants, one, provided that flexibility is provided to allow either stage to receive raw sludge in emergencies; number of digesters in each stage of large plants will be dictated by economics;

TABLE 16.1

TYPICAL SLUDGE QUALITIES AND GENERATION RATES FOR DIFFERENT UNIT PROCESSES (3.)

UNIT PROCESS	LIQUID SLUDGE	SOLIDS CONCENTRATION		VOLATILE SOLIDS	DRY SOLIDS	
	(L/m^3) (1.)	Range (%)	Average (%)	(%)	(g/m^3) (2.)	(g/cap.)
PRIMARY SEDIMENTATION WITH ANAEROBIC DIGESTION						
Undigested (No P Removal)	2.0	(3.5-8)	5.0	65	120	55
Undigested (With P Removal)	3.2	(3.5-7)	4.5	65	170	77
Digested (No P Removal)	1.1	(5-13)	6.0	50	75	34
Digested (With P Removal)	1.6	(5-13)	5.0	50	110	50
PRIMARY SEDIMENTATION AND CONVENTIONAL ACTIVATED SLUDGE WITH ANAEROBIC DIGESTION						
Undigested (No P Removal)	4.0	(2-7)	4.5	65	180	82
Undigested (With P Removal)	5.0	(2-6.5)	4.0	60	220	100
Digested (No P Removal)	2.0	(2-6)	5.0	50	115	52
Digested (With P Removal)	3.5	(2-6)	4.0	45	150	68
CONTACT STABILIZATION AND HIGH RATE WITH AEROBIC DIGESTION						
Undigested (No P Removal)	15.5	(0.8-2.8)	1.1	70	170	77
Undigested (With P Removal)	19.1	(0.8-2.8)	1.1	60	210	95
Digested (No P Removal)	6.1	(1-3)	1.9	70	115	52
Digested (With P Removal)	8.1	(1-3)	1.9	60	155	70
EXTENDED AERATION WITH AERATED SLUDGE HOLDING TANK						
Waste Activated (No P Removal)	10.0	(0.8-1.9)	0.9	70	90	41
Waste Activated (With P Removal)	13.3	(0.8-1.9)	0.9	60	120	55
Sludge Holding Tank (No P Removal)	4.0	(0.8-4.5)	2.0	70	80	36
Sludge Holding Tank (With P Removal)	5.5	(0.8-4.5)	2.0	60	110	50

NOTES :

1. (L/m^3) denotes litres of liquid sludge per cubic metre of treated sewage.
2. (g/m^3) denotes grams of dry solids per cubic metre of treated sewage.
3. The above values are based on typical raw sewage with Total BOD_5 = 170 mg/L, Soluble BOD_5 = 50%, SS = 200 mg/L, P = 7 mg/L, NH_4^+ = 20 mg/L.

Volatile solids loading to primary digester
- 650-1600 g/m^3d;

Nominal minimum hydraulic retention time in primary digester
- 15 days (theoretical SRT requirement of slowest methane producers is approximately 10 days);

Mixing
- thorough mixing via digester gas (compressor power requirement 5 to 8 W/m^3) or mechanical means (6.6 W/m^3) in the primary stage will be necessary in all cases when digesters are proposed; digester mixing studies are now being carried out to more precisely determine mixing requirements;

Heating
- heating must, at least, be provided for the primary digester so that a temperature of 35°C can be maintained. External heat exchanger systems are preferred. Heating should be via a dual-fuel boiler system using digester gas and natural gas, or oil;

Digester covers
- to provide gas storage volume and to maintain uniform gas pressures, a separate gas storage sphere should be provided or at least one digester cover should be of the gas-holder floating type; if only one floating cover is provided, it should be on the secondary digester; insulated pressure

and vacuum relief valves and flame traps should be provided; access manholes and sampling wells should also be provided on the digester covers.

Steel is the most commonly used material for digester covers. However, other properly designed and constructed materials are also successfully employed such as concrete and fibreglass.

Secondary digester sizing

- the secondary digester should be sized to permit solids settling for decanting and solids thickening operations, and in conjunction with possible off-site facilities, to provide the necessary digested sludge storage; the necessary total storage time will depend upon the means of ultimate sludge disposal, with the greatest time required with soil conditioning operations (winter storage), and with less storage required with landfilling or incineration ultimate disposal methods; offsite storage in sludge lagoons, sludge storage tanks, or other facilities, may be used to supplement the storage capacity of the secondary digester; if high-rate primary digesters are used and efficient dewatering within the secondary digester is required, the secondary digester must be conservatively sized to allow adequate solids separation (secondary to primary sizing ratios of 2:1 to 4:1 are recommended);

Sludge piping

- maximum flexibility should be provided in terms of sludge transfer from primary and secondary treatment units to the digesters, between the primary and secondary digesters, and from the digesters to subsequent sludge handling operations; minimum diameter of sludge pipes should be 100 mm; provision should be made for flushing and cleaning sludge piping; sampling points should be provided on all sludge lines; main sludge transfer lines should be from the bottom of the primary digester to the mid-point of the secondary digester; additional transfer lines should be from intermediate points in the primary digester (these can be dual-purpose supernatant and sludge lines);

Supernatant piping

- supernatant should be returned to the treatment plant with flexible points of return to the aerated grit tank, upstream of the primary settling tanks, or to the aeration tank; multiple draw-off points or adjustable supernatant draw-offs, and sampling lines with sample sinks should be provided; both primary and secondary digesters should be equipped with supernating lines so that during emergencies the primary can be operated as a single stage process; precautions must be taken to avoid loss of digester gas through supernatant piping; additional BOD_5 load caused by supernatant return should be considered in aeration system design;

Access bulkhead

- at least one access bulkhead is required through the wall of a digester (see [43]);

Gas piping systems

- refer to [43];

Electrical

- electrical equipment and lighting in hazardous areas in digester control buildings must be Group "D" Class "1";

Overflows

- each digester should be equipped with an emergency overflow system (see [43]).

16.3 AEROBIC DIGESTION

Aerobic digesters treating waste activated sludge should be designed in accordance with the following criteria. If primary sludge is to be included, minimum sludge age and air requirements may have to be increased.

Number of stages

- two;

Number of tanks in each stage

- generally one, unless economics permit more;

Loading

- 1600 $g/m^3.d$ volatile suspended solids based upon first stage volume only;

Sizing

- designed to achieve a minimum sludge age of 45 days, including both stages and sludge age of waste activated sludge; if a total of 45 days sludge age is all that is provided, it is suggested that 2/3 of the total digester volume be in the first stage and 1/3 be in the second stage; actual storage requirement will depend upon ultimate disposal operation (see Anaerobic Digestion subsection); any minor additional storage requirements may be made up in the second stage digester, but if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters (see Section 16.2 for a discussion of separate storage facilities, but designers are cautioned that aerobically digested sludges have greater odour producing potential than anaerobically digested sludges);

Air and mixing requirements

- aeration rate will depend upon the oxygen uptake rate at the maximum solids content experienced; as a guideline, 0.85 $L/m^3.s$ (Litres of air per cubic metre of aeration tank per second) should be provided for diffused aeration systems; a minimum bottom velocity of 0.25 m/s should be maintained while aerating; mechanical surface aeration systems are not recommended due to increased heat loss causing icing problems; air supply to each tank should be

separately valved to allow aeration shut-down in either tank; diffuser type should not be susceptible to plugging during frequent shut-down periods; diffuser drop pipes should be able to withstand impact of ice masses and should allow easy removal for diffuser maintenance;

Tank design

- generally open; tankage should be of commonwall construction or earthen-bermed to minimize heat loss; tank depths 3.6-4.6 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths to, or from both the primary and secondary digester.

SECTION 17

17.0 SLUDGE THICKENING AND DEWATERING

As previously mentioned, a draft MOE policy and its draft guidelines cover sludge stabilization and/or dewatering requirements for the various sludge utilization/disposal options. MOE Regional staff should therefore be consulted to determine the dewatering requirements for any planned sludge utilization/disposal operation.

The sludge solids concentrations which are listed in Table 16.1 are the concentrations which can generally be achieved without the use of separate thickening or dewatering facilities. To achieve any significantly higher concentrations, sludge thickening and/or dewatering facilities will be required.

Sludge thickening normally refers to the process of reducing the free water content of sludges, whereas, dewatering refers to the reduction of floc and capillary water content of sludges. See Tables 17.1 and 17.2 for typical performance expectations for various thickening and dewatering processes.

Before discussing the various unit processes involved, the benefits which can be derived from reductions in sludge water content should be considered. The following lists the main benefits:

- reduction in digester sizing requirements to achieve the same solids retention time;

- reduction in heat exchange capacity requirements;

- reduction in sludge pumpage and transportation costs;

- reduction in ultimate disposal costs;

- reduction of handling problems and leachate production during sludge landfilling operations.

There may be some disadvantages to excessive reduction in water content, as follows, which must also be taken into consideration:

- sludge mixing and blending facilities may be required to combine sludges of differing water content for subsequent treatment operations;

- sludge at 12 to 15 per cent is not free flowing and may require special sludge handling equipment;

- dry sludges because of their significant loss in plant-available nitrogen content may not be as acceptable for spreading on agricultural lands as liquid sludges are.

Wherever possible, pilot-plant and/or bench-scale data should be used for the design of sludge thickening and dewatering facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

In considering the need for sludge thickening and dewatering facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

Besides the information contained in the following subsections, references [44], [45], [46], [47], and [48] will be of assistance to designers. References [44] and [45] are particularly helpful in their coverage of design procedures based on bench-scale or pilot-scale testing.

17.1 SLUDGE CONDITIONING

Sludge dewatering, and to a lesser extent sludge thickening operations, are highly dependent upon sludge conditioning for their successful operation. Sludge conditioning not only affects the solids concentration of the thickened or dewatered sludge, but also affects the solids capture efficiency of the process.

There are several characteristics of sludges which adversely affect attempts to achieve solid-liquid separation. The presence of colloidal particles increases the specific resistance of the sludge and adversely affects sedimentation processes. The net negative charge exhibited by most sewage sludges tends to make the particles repulse each other and thus resist agglomeration into larger particles. Finally, sludge particles have a bound water

content which, if retained, results in low cake solids after solid-liquid separation. Sludge conditioning operations attempt to alter one, or more, of the above sludge characteristics so as to improve the efficiency of the solid-liquid separation processes.

There are two sludge conditioning approaches that can be used. Sludge can be conditioned by physical methods, such as heat treatment or addition of fly ash, or by chemical methods, involving the addition of either organic or inorganic chemicals.

The method selected will not only differ in its effect on the thickening or dewatering process, but will have different effects on subsequent sludge handling operations and on the sewage treatment process itself.

17.1.1 Physical Methods

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. Heat conditioning can be accomplished by either a non-oxidative or oxidative system. With this process, the sludge is treated at temperatures of 175 to 204°C, pressures of 1700 to 2800 kPa and for detention times of 15 to 40 minutes. The high temperatures cause hydrolysis of the encapsulated water-solids matrix and lysing of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the water-solids separation characteristics.

Although the heat conditioning system has been proven to be an effective sludge conditioning technique for subsequent dewatering operations, the process results in a significant organic loading to the aeration tanks of the sewage treatment plant, if the supernatant is returned to the aeration system, due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50 per cent of the total loading on the aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials such as stainless steel. Scale formation in the heat exchangers, pipes and reactor will require acid washing equipment to be provided.

Heat conditioning, particularly the non-oxidative process, can also result in the production of odorous gases in the decant tank. If ultimate sludge disposal is via incineration, these gases can be incinerated in the upper portion of the furnace (760°C, or higher). If incineration is not a part of the sludge handling process, a catalytic or other type of oxidating unit should be used.

The design requirements for a heat conditioning system should be determined by either batch or small-scale continuous pilot-plants. Through such methods, the necessary level of hydrolysis to produce the desired reduction in the specific resistance of the sludge, and the liquor

characteristics can be determined. Tests can also be made at different temperatures and detention times to determine the most effective full-scale operating conditions.

Another common form of physical conditioning is the addition of admixtures such as fly ash, incinerator ash, diatomaceous earth, or waste paper. These conditioning techniques are most commonly used with filter presses or vacuum filters. The admixtures when added in sufficient quantities produce a porous lattice structure in the sludge which results in decreased compressibility and improved filtering characteristics. When considering such conditioning techniques, the beneficial and detrimental effects of the admixture on such parameters as overall sludge mass, calorific value, etc., must be evaluated along with the effects on improved solids content.

Elutriation, although once widely used as a conditioning technique, is no longer a popular process and will therefore not be covered.

Freezing of sludges has been used successfully in Ontario for water treatment plant sludges, but there are no known systems intentionally using slow freezing as a conditioning method for sewage sludges. The process reportedly produces good results for subsequent gravity dewatering of the thawed sludge (up to 16 per cent solids for excess activated sludge and up to 25 per cent solids for digested-sludge). The disadvantages of the system are the high BOD_5 of the effluent and the high cost of the process unless natural means of freezing can be used [51].

17.1.2 Chemical Methods

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. Chemical conditioning for thickening operations attempts to promote more rapid phase separation, higher solids concentration and a greater degree of solids capture. With dewatering operations, chemical conditioning is used in an attempt to enhance the degree of solids capture by destabilization and agglomeration of fine particles. This promotes the formation of a cake which then becomes the true filter media in the dewatering process.

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are polymers. For dewatering by vacuum filtration, ferric salts, often in conjunction with lime, are most commonly used, although with centrifuge dewatering, chemical conditioning using polymers is most prevalent, with metal salts being avoided mainly due to corrosion problems. For dewatering by filter presses, the use of high molecular weight polymers, in lieu of lime and ferric chloride, for sludge conditioning has been successfully employed in the United Kingdom and Europe. This innovative sludge dewatering system is now favourably considered in Ontario. In fact, the installation of such a system is presently underway at two sewage treatment plants. The ultimate disposal methods may also have an effect on the choice of conditioning chemicals. For instance, lime and ferric compounds should be avoided with incineration options.

The selection of the most suitable chemical(s) and the actual dosage requirements for sludge conditioning can best be determined by full-scale testing.

Laboratory testing should, however, be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15 per cent of full-scale needs. The previously mentioned references should be referred to for more information on testing technique.

17.2 SLUDGE THICKENING

Sludge thickening can be employed in the following locations in a sewage treatment plant:

- prior to digestion for raw primary, excess activated sludge or mixed sludges;
- prior to dewatering facilities;
- following digestion for sludges or supernatant;
- following dewatering facilities for concentration of filtrate, decant, centrate, etc.

The commonly employed methods of sludge thickening and their suitability for the various types of sludge are shown in Table 17.1. In selecting a design figure for the thickened sludge concentration, the designer should keep in mind that all thickening devices are adversely affected by high SVI's and benefited

TABLE 17.1

SLUDGE THICKENING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

THICKENING METHOD	SLUDGE TYPE	PERFORMANCE EXPECTED
GRAVITY	Raw Primary	Good, 8-10% Solids
	Raw Primary and Waste Activated	Poor, 5-8% Solids
	Waste Activated	Very Poor, 2-3% Solids (Better results reported for oxygen excess activated sludge)
	Digested Primary	Very Good, 8-14% Solids
	Digested Primary and Waste Activated	-
DISSOLVED AIR FLOTATION	Waste Activated (Not generally used for other sludge types)	Good,4-6% Solids and $\geq$ 95% Solids Capture With Flotation Aids.
CENTRIFUGATION	Waste Activated	8-10% and 80-90% Solids Capture with Basket Centrifuges; 4-6% and 80-90% Solids Capture with Disc-nozzle Centrifuges; 5-8% and 70-90% Solids Capture with Solid Bowl Centrifuges.

by low SVI's in the influent activated sludges. The ranges of thickened sludge concentrations given in Table 17.1 assume an SVI of approximately 100.

Wherever thickening devices are being installed special consideration must be given to the need for sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines, and thickening equipment. Also, where thickeners are to be housed, adequate ventilation will be required.

17.2.1 Gravity Thickening

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone. Due to the better performance of other methods for waste activated sludges, gravity thickening has limited application for such sludges.

Gravity thickeners should be designed in accordance with the following parameters:

Tank shape
- circular;

Tank depth (SWD)
- 3 to 3.7 m;

Tank diameter
- up to 21 - 24 m;

Floor slope
- acceptable range 2:12 to 3:12;

Solids loadings

- primary sludges 96 to 120 $kg/m^2.d$;
- waste activated 12 to 36 $kg/m^2.d$;
- combination of primary and waste activated
- based on weighted average of above loading rates;
- use of metal salts for P removal may affect solids loading rates;

Overflow rate

- to prevent septic conditions, an overflow rate of 0.19 to 0.38 $L/m^2.s$ is generally designed for;

Mechanical rake

- rake should have a tip speed of 50 to 100 mm/s;
- to be equipped with hinged-lift mechanisms when handling heavy sludges such as lime treated primary sludges, otherwise optional;
- surface skimmer is recommended;

Sludge underflow piping

- keep length of suction lines as short as possible;
- dual sludge withdrawal lines should be considered;

Chemical conditioning

- provision should be made for the addition of conditioning chemicals into the sludge influent lines (polymers, ferric chloride or lime are the most likely chemicals to be used to improve solids capture);

Sludge volume ratio (SVR)

- volume of sludge blanket divided by volume of sludge withdrawn daily should be 0.5 to 2 days.

17.2.2 Air Flotation

Unlike heavy sludges, such as primary and mixtures of primary and excess activated sludges, which are generally most effectively thickened in gravity thickeners, light excess activated sludges can be successfully thickened by flotation.

The advantages of air flotation compared with gravity thickeners for excess activated sludges include its reliability, production of higher sludge concentrations, and better solids capture. Its disadvantages include the need for greater operating skill and higher operating costs.

Experience has shown that flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters and some bench-scale and/or pilot-scale testing will be necessary. The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operation:

Tank dimensions

- vary with suppliers;

Air buoyancy systems

- vary with suppliers;

Air to solids weight ratio

- 0.02 to 0.05;

Recycle ratios

- vary with suppliers (0 to 500%);

Solids loadings (with waste activated sludge to achieve 4% float solids)

- 48 $kg/m^2.d$ (without flocculating chemicals);
- up to 240 $kg/m^2.d$ (with flocculating chemicals);

Chemical conditioning

- feed chemical to mixing zone of sludge and recycled flow;
- most installations now use chemical conditioning with polymers to achieve more economic operation;
- polymer fee range 0 to 25 g/kg of dry solids;

Hydraulic feed

- up to 1.74 $L/m^2.s$ (based on total flow including recycle, when polymers used);
- without chemicals, lower rate must be used;
- feed rate should be continuous rather than on-off;

Detention time

- not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket;

Thickened sludge withdrawal

- surface skimmer moves thickened sludge over dewatering beach into sludge hopper;
- either positive displacement, or centrifugal pumps which will not air bind should be used to transfer sludge from hopper to next phase of process;
- in selecting pumps, maximum possible sludge concentrations should be taken into consideration;

Bottom sludge

- a bottom collector to move draw off settled sludge into a hopper must be provided;
- draw off from the hopper may be by gravity or pumps.

17.2.3 Centrifugation

To date, there has only been limited application of centrifuges for sludge thickening despite their common use for sludge dewatering. As thickening devices, their use have been generally restricted to excess activated sludges. Three types of centrifuges have been used with such sludges - the solid-bowl decanter, disc-nozzle and basket types.

In the way of general comments, the following are given:

- centrifugal thickening operations can have substantial maintenance and operating costs;

- where space limitations, or sludge characteristics make other methods unsuitable, or where high-capacity mobile units are needed, centrifuges have been used;

- thickening capacity, thickened sludge concentration and solids capture of a centrifuge is greatly dependent on the SVI of the sludge;

- 85 to 95 per cent solids recovery will generally be the most suitable operating range;

- polymer feed range 0 to 4.0 g/kg of dry solids;

- early experience with disc-nozzle type centrifuges found clogging of the sludge discharge nozzles to require frequent maintenance; recent use of rotary screens and cyclones have helped alleviate these problems;

- basket type centrifuges have seen limited use; due to their low capacities and batch operations their use has been generally restricted to small plants.

17.3 SLUDGE DEWATERING

Sludge dewatering will often be required at sewage treatment plants prior to ultimate disposal of sludges. Since the processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major

influence on the dewatering method most suitable for a particular sewage treatment plant. Also of influence will be the characteristics of the sludge requiring dewatering, that is, whether the sludge is raw or digested, whether the sludge contains waste activated sludge, or whether the sludge has been previously thickened. With raw sludge, the freshness of the sludge will have a significant effect on dewatering performance (septic sludge will be more difficult to dewater than fresh raw sludge).

Table 17.2 gives the solids capture, solids concentrations normally achieved, energy requirements and suitable ultimate disposal options for various dewatering methods. The solids concentrations shown in Table 17.2 assume that the sludges have been properly conditioned. Designers should be aware that phosphorus removal chemicals (alum or ferric chloride) will reduce allowable solids loading rates for dewatering equipment and produce a lower cake solids concentration than would be expected without phosphorus removal.

Due to the fact that the ammonium nitrogen content of sludges is largely associated with the liquid fraction of sewage sludges and the acceptability of sludges for spreading on agricultural land relates to minimum ratios of nitrogen to heavy metal concentrations, dewatered sludges will generally be less desirable for final spreading on agricultural lands than liquid sludges. To enable sludges to be handled and spread as liquids, the upper limit for solids content will generally be in the order of 12 per cent. This would leave only

TABLE 17.2

SLUDGE DEWATERING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

DEWATERING METHOD	SOLIDS CAPTURE (%)	SOLIDS CONCENTRATIONS NORMALLY ACHIEVED (1.)	MEDIAN ENERGY REQUIRED (MJ/dry tonne) (2.)	SUITABLE ULTIMATE DISPOSAL METHODS		
				LANDFILL	AGRICULTURAL UTILIZATION	INCINERATION
VACUUM FILTER	90-95	Raw Primary + WAS (10-25%)	1000	YES	NO	YES
		Digested Primary + WAS (15-20%)		YES	NO	YES
		WAS (8-12%)		YES	NO	NO
FILTER PRESS	90-95	Raw Primary + WAS (30-50%)	350	YES	NO	YES
		Digested Primary + WAS (35-50%)		YES	NO	YES
		WAS (25-50%)		YES	NO	YES
CENTRIFUGE (SOLID BOWL)	95-99	Raw or Digested Primary + WAS (15-25%)	350	YES	NO	YES
		WAS (12-15%)		YES	NO	NO
BELT FILTER	85-95	Raw or Digested Primary + WAS (14-25%)	130	YES	NO	YES
		WAS (10-15%)		YES	NO	NO

NOTES :

1. Including conditioning chemicals, if required.
2. MJ/dry tonne - denotes megajoules per dry tonne of sludge throughout.

thickened sludges acceptable for spreading on agricultural land by liquid spreading techniques, provided they still meet the guideline criteria for liquid sludges (nitrogen to metal ratios).

The required solids concentration for sludges which are to be landfilled at sanitary landfill sites will be influenced by the quantities of sludge to be disposed of in relation to the quantities of municipal refuse, characteristics of the site itself, and the expected effects of the liquid addition to the site. With small quantities of sludge for co-disposal landfilling with garbage, liquid sludge at solids concentrations as low as 3 per cent may be acceptable. For sludge only landfill operations, a minimum of 15 per cent solids concentration is generally required to support cover material. If sludge is to be disposed of in sludge lagoons, dewatering may not be necessary unless it is justifiable for economic reasons relating to haulage costs.

For ultimate disposal by incineration, sludges should ideally be concentrated to a solids concentration where they will burn autogenously. This solids concentration will vary somewhat with sludge type, volatile solids percentage, and the chemical composition of the solids, but a minimum concentration in the order of 30 per cent will generally be required. With chemical conditioning, autogenous sludge solids concentrations can generally only be attained using filter presses. With conditioning by heat treatment, sludge dewatering methods such as filter presses, belt filter presses, centrifuges

and perhaps even vacuum filters will be capable of producing autogenous sludge solids concentrations.

As with thickening systems, dewatering facilities may require sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines and plugging or damaging dewatering equipment. Also, adequate ventilation equipment will be required in buildings housing dewatering equipment.

In evaluating dewatering system alternatives, the designer must consider the capital and operating costs, including labour, parts, chemicals and energy, for each alternative as well as for the effects which each alternative will have on the sewage treatment and subsequent sludge handling and ultimate sludge disposal operations. Since labour and particularly energy costs are escalating at a rapid rate, it is suggested that these annual costs be converted to capital cost equivalents for evaluation purposes. The suggested method for capitalizing these costs is outlined in the report "Guidelines for Energy Conservation in the Design of Sewage Systems and Treatment Facilities in the Province of Ontario", August, 1977 [21].

17.3.1 Vacuum Filters

Rotary vacuum filters have been the most commonly used mechanical systems for sludge dewatering in Ontario to date. With the recent developments of new, lower cost and more effective dewatering systems the use of vacuum filters is beginning to decline.

Rotary drum, rotary belt and spring coil variations of the rotary vacuum filter are available for use. The primary machine variables which affect dewatering are vacuum pressure, drum submergence, drum speed, degree of sludge agitation and filter medium. The operation variables which affect dewatering performance are sludge type, sludge conditioning and sludge characteristics including initial solids concentration, nature of sludge solids, chemical composition, sludge compressibility, sludge age, temperature and filtrate viscosity.

Of primary importance with vacuum filters is the solids concentration of sludge fed to the units. With all other operating variables remaining constant, increases in filtration rates vary in direct proportion to feed solids. Sludge thickening prior to vacuum filters is therefore extremely important. Higher concentrations in the sludge feed also result in lower filtrate solids.

Vacuum filtration systems should be designed in accordance with the following parameters:

Sludge feed pumps

- variable capacity;

Vacuum pumps

- generally one per machine with capacity of 10 $L/m^2.s$ at 65 kPa, or more, vacuum;

Vacuum receiver

- generally one per machine; max. air velocity 0.8 to 1.5 m/s; air retention time 2-3 minutes; filtrate retention time 4-5 minutes; all lines to slope downward to receiver from vacuum filter;

Filtrate pumps

- generally self-priming centrifugal; suction capacity greater than vacuum pump (65 to 85 kPa vacuum); with flooded pump suctions; with check valve on discharge side to minimize air leakage into the system; pumps must be sized for the maximum expected sludge drainage rates (usually produced by polymers);

Sludge flocculation tank

- constructed of corrosion-resistant materials; with slow speed variable drive mixer, detention time 2-4 minutes with ferric and lime (with polymers shorter time may be used);

Wash water

- filtered final effluent generally used;

Sludge measurement

- should be provided unless measured elsewhere in plant;

Solids loading rate

- 7-14 $g/m^2.s$ for raw primary; 2.75-7 $g/m^2.s$ for raw primary + WAS; 4-7 $g/m^2.s$ for digested primary + WAS; not considered practical for use with WAS alone.

17.3.2 Filter Presses

Recent changes in the design of filter presses, including elimination of leakage problems, more automation, improved filter media, greater unit capacities and development of high molecular weight polymers and compatible polymer feed systems, have resulted in renewed interest in this method of sludge dewatering.

Variations in the filter process which are now available on the market include units with recessed plates or plates with frames, top or central sludge feed, air pressure assisted sludge cake release, automatic washing of filter media, sequential or simultaneous release of sludge cake, final compression stage using flexible diaphragm behind filter media, etc.

The primary advantage offered by filter presses is the ability to concentrate all types of waste water sludges to very high concentrations. Concentrations as high as 45 to 50 per cent including conditioning chemicals, can generally be achieved with properly conditioned sludges. Filter presses are also able to effect high efficiencies in solids capture and as a result produce relatively clear filtrate. Their primary disadvantages are that the process is batch rather than continuous, cake removal still requires some manual assistance, and large quantities of conditioning agents are generally necessary.

As with vacuum filters, the capacity of filter presses is greatly affected by the initial solids concentration. With low feed solids, chemical requirements increase significantly.

Sludge thickening should therefore be considered as a pre-treatment step. The sludge is generally conditioned with a physical conditioning agent such as fly ash or with chemicals such as ferric chloride, lime, and alum, although use of polymer conditioning agents is becoming more common with the development of compatible polymer feed systems. In some instances, precoats are applied to the filter media prior to the addition of sludges to prevent premature media blinding. Various materials can be used for precoat including diatomaceous earth, fly ash, incinerator ash and various types of industrial waste by-products.

Filter press systems should be designed in accordance with the following guidelines:

Sludge conditioning tank

- detention time maximum 20 minutes at peak pumpage rate;

Feed pumps

- variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge; pumps should be of a type to minimize floc shear; pumps must deliver high volume at low head initially and low volume at high head during latter part of cycle; ram or piston pumps, progressing cavity pumps or double diaphragm pumps are generally used;

Cake handling

- filter press must be elevated above cake conveyance system to allow free fall; cake can be discharged directly to trucks, into dumper boxes, or onto conveyors (usually

belt or drag chain type); conveyors must be able to withstand impact of sludge cakes; cable cake breakers may be needed;

Cycle times
- 1.5 to 6 h (normally 1.5 to 3 h).

Operating pressures
- usually 700 to 1400 kPa, but may be as high as 1750 kPa;

Operating pressures depend on the different types of presses and the chemical agents used for sludge conditioning. These pressures may be developed either hydraulically or by a combination of hydraulic and pneumatic means. For example, recessed plate filter presses with diaphragm membranes for dewatering polymer conditioned sludges are first brought to approximately 700 kPa pressure hydraulically (pumping) and then the membranes are inflated pneumatically to provide a final squeezing pressure of approximately 1050 kPa.

While the magnitude of pressure applied does not adversely affect the dewatering process, if lime and ferric chloride are used as sludge conditioning, it is very important that the generating pressure shall not exceed 1000-1050 kPa if polymer is applied as the conditioning agent.

17.3.3 Centrifuges

The centrifuge types which have been used for waste water sludge dewatering include the solid bowl, basket and disc centrifuges. The most frequently used is the continuous countercurrent

solid bowl centrifuge. Disc centrifuges have seen more application for thickening rather than dewatering. Due to the potential for plugging of the nozzles of disc centrifuges, they must be preceded by efficient degritting and screening facilities. Due to their infrequent use for dewatering and their inherent plugging problems, disc centrifuges, although capable of reaching the lower end of dewatering solids concentrations, are generally only used for thickening operations and will not be discussed further in this dewatering section.

17.3.3.1 Solid Bowl Centrifuges

The machine variables of importance for dewatering include bowl length/diameter ratio, bowl angle, bowl flow pattern, bowl speed, pool volume, conveyor design and relative conveyor speed.

Bowl length/diameter ratios of 2.5 to 4.0 are usually provided to ensure adequate settling time and surface area. Bowl angles must be kept shallow.

The bowl flow pattern can be either countercurrent or concurrent. By using concurrent flow, the settled sludge is not disturbed by the incoming feed and turbulence is reduced. The disadvantages of concurrent flow are the need for a long feed tube and the long travel distance needed to remove the sludge. Other proprietary feed inlets have also been developed to minimize the disturbance to the previously settled solids.

Increased bowl speed increases the centrifugal forces available for clarification, but the settled solids become more difficult to remove due to the higher g-forces. Increased bowl speed, however, will also increase abrasion damage within the centrifuge, noise and vibration. Lower speed machines have been developed which achieve high solids capture. Sludge inlet conditions with these low speed machines have also been improved to minimize the acceleration and turbulence. These low speed machines have lower noise levels, minimized internal wear and have lower power requirements, but may have increased conditioning chemical requirements.

Retention time in the centrifuge will increase with increases in pool volume. With longer retention times achieved by greater pool depth, solids capture increases, but cake solids concentrations will decrease due to reduced retention time on the drying deck and also due to the capture of finer solids with higher moisture content. Pool depth can be varied by adjustable weirs.

Conveyor design and speed will affect the efficiency of solids removal. Differential speed must be kept low enough to minimize turbulence and internal wear yet high enough to provide sufficient solids handling capacity. The most suitable conveyor pitch will be affected by the characteristics of the sludge to be handled. With coarse, high strength solids, conveyors with high pitch angles can be used, but with fine solids low pitch angles must be used. Conveyor differential speed can often be "fine-tuned" following installation.

Important process variables affecting the centrifuge efficiency are feed rate, feed consistency, temperature and the chemical coagulants used.

As hydraulic flow rate increases through a centrifuge, solids capture decreases and cake solids concentrations increase due to the loss of fines in the centrate. If the feed solids are too high, solids buildup within the bowl can take place reducing clarification volume. Both solids and hydraulic overloading can, therefore, occur.

For most waste water sludges, the capacity of the centrifuge will be limited by the clarification capacity (hydraulic capacity) and therefore the solids concentration. Increasing the feed solids will increase the solids handling capacity. Thickening should, therefore, be considered as a pre-treatment operation.

Since temperature affects the viscosity of sludges, if the temperatures will vary appreciably (as with aerobic digestion), the required centrifuge capacity should be determined for the lowest temperature expected.

The chemical conditioning agents most commonly used with centrifuges are polymers. Flocculant is generally directly injected into the interior of the centrifuge to avoid shearing the floc. Maximum effectiveness is generally achieved by diluting the flocculant to a strength of 0.1 per cent, or less.

Other general design guidelines for solid bowl centrifuges are as follows:

Feed pump

- sludge feed should be continuous; pumps should be variable flow type; one pump should be provided per centrifuge for multiple centrifuge systems; chemical dosage should vary with pumpage rate;

Sludge pre-treatment

- depending upon the sewage treatment process, grit removal, screening or maceration may be required for the feed sludge stream;

Solids capture

- 85 to 95 per cent generally desirable;

Machine materials

- generally carbon steel or stainless steel; parts subject to wear should be protected with hard facing materials such as a tungsten carbide material;

Machine foundations

- foundations must be capable of absorbing the vibratory loads;

Provision for maintenance

- sufficient space must be provided around the machine(s) to permit disassembly; an overhead hoist should be provided; hot and cold water supplies will be needed to permit flushing out the machine; drainage facilities will be necessary to handle wash water.

17.3.4 Belt Filter Presses

Belt filter presses, originally developed in Europe are gaining popularity here. Although variations in the process exist, the system basically consists of two continuous, separate belts - a press belt and a filter belt. Sludge is confined between the two belts with the press belt exerting pressure on the filter belt, thereby continuously dewatering the sludge.

There are generally three distinct dewatering zones throughout the process. The first zone is a gravity draining zone, the second is a pressure zone and the third is a shear zone. Pressure is exerted by the rollers, conveying belts, or other external devices. In the shear zone, the sludge cake is further dewatered by deforming the sludge cake by passing the belts around rolls and/or between vertically offset rollers causing a serpentine configuration in the sludge cake movement.

Most types of waste water sludges can be dewatered with belt filter presses and the results achieved are generally superior to those of vacuum filters. Belt filter presses generally use only 1/3 the power requirements of vacuum filters, and do not experience sludge pickup problems often encountered with vacuum filters. Belt filter presses have been reportedly used to further dewater the sludge cake from vacuum filters with excellent results. Such a method should be considered for upgrading existing dewatering systems.

Chemical conditioning is generally with polymers.

Solids handling capabilities are likely to range from 50 g/m.s of dry solids (based on belt width) for excess activated sludge to 330 g/m.s for primary sludge. Expected solids concentration results are shown in Table 17.2. Solids capture is usually in excess of 85 per cent and often as high as 95 per cent.

17.4 SLUDGE DRYING BEDS

Conventional sand drying beds due to their low capital cost, simple operation and capability of producing high solids concentrations (greater than 40 per cent), should be considered as a sludge dewatering alternative, especially for small to medium-sized sewage treatment plants. Due to the presence of sludge drying beds at a large number of existing Ontario sewage treatment plants, their use as an emergency sludge dewatering technique to backup mechanical dewatering processes should also be considered.

Since sludge conditioning can reduce the required drying time to 1/3 or less, of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

Some of the advantages and disadvantages of using sludge drying beds are as follows:

(a) Advantages

- low capital cost, if land is readily available

- process simple to operate;

- low energy consumption;

- less sensitive to feed solids concentration and other variations in sludge characteristics;

- higher cake solids capability than most mechanical systems;

(b) <u>Disadvantages</u>

- may produce odours if sludge not adequately digested, or otherwise stabilized;

- requires considerable land area;

- cycle times are very dependent on climatic conditions;

- labour extensive if manual sludge removal technique used;

17.4.1 <u>Design Parameters</u>

The usual design parameters for conventional sand drying beds are as follows:

- drainage tile 100 mm diameter, or more, spaced 2.4 to 3.0 m apart, with slope of one per cent, or more;

- bottom of cell should be of impervious material such as clay or asphalt;

- drainage tile bedded in gravel layer usually 200 mm to 500 mm thick, graded from 25 mm on the bottom to 3 mm on the top;

- sand layer above gravel usually 250 to 450 mm thick with an effective size of 0.3 to 1.2 mm and a uniformity coefficient of less than 5.0;

- bed size generally 4.5 to 7.5 m wide with length selected to satisfy desired bed loading volume;

- dosing depth generally 200 to 300 mm for warm weather operating modes; for winter freeze drying depths of 1 to 3 m can be used depending upon number of degree days in winter [51];

- inlet pipe usually one per cell, with inlet 300 mm above bed surface and with splash pad to prevent bed disruption and to promote even distribution of sludge; provision for flushing inlet lines should be provided;

- usually 3 beds desirable for flexibility of operation;

- sludge removal can either be manual or mechanical; if mechanical, concrete vehicle tracks generally required with clay tiles, but may not be necessary with perforated plastic pipe and/or flotation-tire equipped front-end loader;

- outer walls and partition walls should be water-tight;

- underdrains should discharge back to the secondary treatment section of the sewage treatment plant;

- recommended sizing for uncovered beds between latitudes 40 to 45°N is 0.16 m^2/cap and for north of 45°N, 0.20 m^2/cap (for primary plus waste activated sludge following anaerobic digestion; the recommended sizing for covered beds with the same sludge type is 0.13 and 0.16 m^2/cap, respectively;

Other types of sludge drying beds which have been used in the United States include the following:

- paved rectangular beds with a centre sand drainage strip, with or without heating and with or without covering;

- "wedge-wire" drying beds with a wedge wire system, provision for an initial flood with a water layer, followed by sludge introduction on top of water layer, controlled cake formation and provision for controlled underdrainage and mechanical sludge removal;

- rectangular vacuum assisted sand beds.

For more information on experience in the United States with the above bed types, refer to references [47] and [48].

SECTION 18

18.0 SLUDGE LAGOONS

Two types of sludge lagoon systems have been most commonly constructed in Ontario - thickening lagoons and sludge transfer site lagoons.

Thickening lagoons have generally been built at or near the site of the sewage treatment plants so that the sludge can be conveyed to the lagoons by pumpage or gravity and so that the supernatant can be returned to the sewage treatment plant for further treatment. These lagoons require approval under the OWR Act since they form part of the sewage treatment works.

Sludge transfer site lagoons have usually been built as temporary sludge storage facilities, which are required with a program for sludge utilization on agricultural land, to hold liquid sludge delivered by a tank truck during times of the year when spreading on agricultural land cannot be carried out due to such factors as wet ground, frozen ground or snow cover. With true transfer site lagoons, no particular attempt is made to withdraw supernatant or to otherwise thicken the sludge beyond the natural thickening that occurs due to evaporation and minor exfiltration of water from the lagoon cell. Transfer site lagoons require approval under the Environmental Protection Act.

In some circumstances where suitable land surrounds the lagoon, a combination thickening and transfer lagoon can be built where supernatant can be spray irrigated onto the

surrounding land and the thickened sludge can then be hauled away for spreading on farmland. Withdrawal of supernatant will result in increases in sludge concentration to the extent that sludge removal by pumpage may become difficult, or impossible. Above a solids concentration of 7 to 8 per cent, pumpage can become difficult. In addition, since the ammonia concentration of the sludge is largely contained in the liquid fraction, supernatant withdrawal may drop the ammonium plus nitrate nitrogen to metal ratios to unacceptable levels for spreading on farmland.

18.0.1 Design Considerations

The design and location of sludge lagoons must take into consideration many factors, including the following:

- Possible nuisances - odours, appearance, mosquitos;

- Design - number, size, shape and depth;

- Loading factors - solids concentration of digested sludge, loading rates;

- Soil conditions - permeability of soil, need for liner, stability of berm slopes, etc.;

- Groundwater conditions - elevation of maximum groundwater level, direction of groundwater movement, location of wells in the area;

- Sludge and supernatant removal - volumes, concentrations, methods of removal, method of supernatant treatment and final sludge disposal;

- Climatic effects - evaporation, rainfall, freezing, snowfall, temperature, solar radiation.

See references [53] and [54] for further discussions of sludge lagoon design. References [51] and [52] may be consulted for a discussion of natural freeze-thaw sludge conditioning as a method to improve sludge dewatering.

Within the United States, a more rational approach to the design of sludge lagoons has been practiced in recent years. References [20] outlines the design procedures for facultative sludge lagoons where sludge loading rates are limited and mechanical aeration is used to develop an aerobic surface layer to minimize odour generation problems. Such lagoon systems have apparently been built as far north as Corvallis, Oregon and have operated successfully under freezing conditions.

SECTION 19

19.0 WASTE STABILIZATION PONDS

19.1 WHERE APPLICABLE FOR USE

Waste stabilization ponds (lagoons) are often capable of providing the required degree of sewage treatment at considerably lower cost than mechanical sewage treatment processes. This will be particularly the case with small municipalities, where sufficient low cost land is available in the vicinity of the service area, and where low permeability soils are available for cell construction.

Other circumstances when waste stabilization ponds have advantages over other sewage treatment methods are where receiving streams experience experience insufficient flows during at least part of the year to provide adequate dilution for continuous discharges, or where downstream recreational water uses make summer effluent discharges undesirable. In both cases, the retention capabilities of waste stabilization ponds make them the preferred sewage treatment system.

19.2 TREATMENT EFFICIENCY

Waste stabilization ponds, when operated on a fill-and-draw basis with phosphorus removal effected by batch dosage with alum, are able to achieve an effluent quality comparable to conventional activated sludge plants with phosphorus removal. See Table 6.1. To achieve such quality, the lagoon cells must be ice free at

the time of planned discharge so that batch dosage can be used for phosphorus removal. Continuous addition of alum to the raw sewage entering lagoon cells has not proven to be as effective a means of reducing phosphorus levels, although total phosphorus reduction to 1 mg/L can be achieved.

Recent research work with waste stabilization ponds has shown that if lagoons are discharged prior to, or too soon after, the ice cover leaves the lagoon in the spring, ammonia and hydrogen sulfide levels in the effluent may be high enough to cause fish kills in the receiving stream. Hydrogen sulfide will naturally dissipate soon after the ice cover melts or it may be stripped from the effluent by aeration. To reduce the risk of odours, it may be oxidized by aeration of the effluent or its production eliminated with continuous aeration throughout the winter months. Ammonia nitrogen will be stripped from the lagoon contents during the summer months of high algal growth.

19.3 TREATMENT AND HOLDING CAPACITY REQUIREMENTS

Before the design of a waste stabilization pond system can be initiated, the designer should contact staff of the Ministry of the Environment's Regional Office to determine the following:

- whether the lagoon can be continuously discharged or must operate on a fill-and-draw basis;

- the period of the year if any, when discharge will not be permitted;

- whether phosphorus removal will be necessary, and if required, to what level;

- whether ammonia and/or hydrogen sulfide concentrations will require special considerations;

- what discharge rates will be permitted with fill-and-draw lagoons and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flow rates;

- what the minimum time for discharge of lagoon cell contents should be for fill-and-draw systems.

19.4 DESIGN CONSIDERATIONS

The design and location of waste stabilization ponds must take into consideration many factors including the following:

- Possible nuisances - odours, appearance, mosquitos (see Appendix B for recommended land use surrounding lagoons);

- Size, shape and depth - acute angles should be avoided; maximum size of cells 8 ha, but 4 ha preferred; maximum operating depth generally 1.8 m, unless preceded by secondary treatment or equivalent; bottom 0.3 m must be retained following discharge;

square cells preferred to long narrow rectangular cells; long dimension(s) should not align with prevailing wind direction;

- Loading factors - BOD_5 loading should not exceed 22 kg/ha.d for significant time periods;

- Soil and groundwater conditions - a soils consultant's report is recommended for all earthen berm construction to demonstrate the suitability of the soils; in certain circumstances, a hydrogeologist's report may be required to assess possible impact on the watertable; a minimum separation of 3.0 m between the cell bottom and bedrock is recommended; cell bottoms should be located at least 1.2 m above high groundwater level to prevent groundwater inflow and/or liner damage;

- Liners - under certain soil circumstances, liners may be required in order to minimize excessive leakage; where clay liners are used, precautions should be taken to avoid erosion and desiccation cracking prior to placing the system in operation;

- Berms - minimum top width 2.4 m (3.0 m preferred where liquid alum trucks must travel berms); minimum freeboard above maximum liquid level 0.6 m; all topsoil must be stripped from the area on which the berms are to be constructed; berm slopes should not exceed 4:1 inside and 3:1 outside unless greater slopes recommended by a soils consultant;

- Surface Treatment - minimum protection of mulch seeding on 0.1 m topsoil layer for all surfaces not normally underwater and for inner berm surfaces down to 1.0 m below the high water level; where cells will be empty for a long period before filling, entire berm surface should be mulch seeded to prevent erosion; cell bottom should be level and flat ($\pm$ 0.15 m); a graveled access road and ramp to the top of the berms should be provided;

- Site drainage - surface drainage must be routed around and away from cells; field tiles within the area enclosed by the berms must be located and blocked to prevent cell content leakage; measures must be taken where necessary to avoid disruption of field tile and surface drainage of adjacent lands by constructing drainage works to convey water around the lagoon site;

- Soil compaction - cell bottoms and berms shall be compacted to at least 90 per cent Standard Proctor Density;

- Number of cells - minimum of two;

- Influent system - the main inlet sewer or forcemain should terminate at a chamber which permits hydraulic and organic load splitting between the lagoon cells; the ability to introduce raw sewage to all cells is desirable, but as a minimum, there must be a capability to divide raw sewage flows between enough cells to reduce the BOD_5 loading to 22 kg/ha.d, or less; inlet

piping should provide a scouring velocity of 0.6 m/s; inlet pipes should terminate with a concrete pad and upturned elbow with pipe extending 0.45 m above the cell bottom; inlet pipes should extend to at least 30 m from the berm centreline; inlet chamber should be provided with a lockable aluminum cover plate or grating, divided into small enough sections to permit easy handling; influent system should be sized to permit peak raw sewage flow to be directed to any one of the primary cells;

- Effluent system - with fill-and-draw operations, must be sized to permit all cells to be discharged in the minimum time specified by MOE Regional staff; with continuous discharge systems the discharge capacity must be at least equal to the expected future peak raw sewage pumpage rate; effluent from each cell should be drawn from 0.3 m above the cell bottom; outlets from continuous discharge lagoons should lead to effluent chamber(s) which permit level regulation; outlet pipes from fill-and-draw lagoons may not require effluent chambers and may be equipped with individual, lockable shut-off valves; discharge flow control may be required, in which case, a flow measurement system will be required; all cells should be provided with an emergency overflow system to overflow when the liquid contents reach within 0.6 m of the top of the berms;

- Cross-connection piping - adjacent cells should be interconnected to permit flow between cells; where cells are at or near

the same elevation, the pipes should be valved; where cell elevations differ significantly, the cross connection pipe should have a chamber with a weir to control flow from the higher cell; the valve or chamber should be provided with suitable locking devices; and be located off the traveled portion of the top of the berm;

- Prevention of short-circuiting - with continuous discharge lagoons short-circuiting must be minimized, especially to avoid need for effluent disinfection; at least two, or more, cells should be provided and designed to permit series (as well as parallel) operation; effluent piping should be as far removed as possible from inlet or cross-connection piping; wind induced currents should be considered when lagoon orientation is being selected;

- Fencing - the lagoon site should be fenced and provided with a locked access gate; lagoon fencing requirements will vary, depending upon the site location, from a minimum of farm fencing if surrounded by agricultural land to more secure fencing if required due to the neighbouring land use; warning signs should be provided along the fence to indicate the nature of the facility, prohibit trespassing and provide emergency phone numbers and/or addresses.

19.5 SOIL CONDITIONS AND LAGOON LEAKAGE

As previously indicated, a soils consultant's report will be required for all earthen berm construction proposals. This report should comment on a number of design aspects including the following:

- the suitability of the native soils for the proposed construction;
- the maximum groundwater elevation;
- the depth to bedrock;
- the soils strata which will be suitable for forming the cell bottom and berm cores and estimates of their permeabilities;
- the soils strata which will require removal from the site and estimates of their permeability;
- the soils strata which will be suitable for topdressing, etc.;
- the need for a liner;
- the berm slopes which will be stable;
- the degree of compaction which should be achieved during construction and any necessary special construction methods which should be followed;
- suggested methods of erosion control;

- the estimated initial clear water leakage rate which should be experienced from the cell structures;

It is an acknowledged fact that all waste stabilization ponds of earthen berm construction experience a certain amount of leakage. The rate of leakage that occurs depends upon the hydraulic gradient to the watertable, the type and permeability of the soils used for construction and what if any soil pore clogging action has taken place since the initial construction. The leakage rate that will be acceptable for any given lagoon system will depend upon a number of factors, including the following:

- the ability of the lagoon to limit leakage sufficiently to result in adequate operating depths to provide the necessary treatment prior to seepage into the ground, and to prevent undesirable weed growths, stagnation and odour problems;

- the direction in which groundwater movement will take place and the location of groundwater uses or groundwater discharge into surface waterbodies;

- the likelihood of surface breakout of seepage from the lagoons either on the berms themselves or elsewhere on or off-site;

- the expected quality of the seepage waste from the cells;

- the assimilation which will occur in the watertable prior to groundwater use or groundwater discharge into a surface waterbody.

The above factors should be evaluated by a hydrogeologist when a lagoon is being considered for a site where leakage is expected to be significant and where there are nearby groundwater uses or surface waterbodies which are likely to be adversely affected.

The natural sealing action which takes place in the soils used for lagoon construction should also be considered. This sealing action results from physical clogging of soil pores by sewage solids, chemical clogging of soil pores by ionic exchange, and biological and organic clogging by microbial growth. Reduction in initial leakage rates from lagoon cells may also occur due to groundwater mounding below the cells resulting in reduced hydraulic gradients. The initial seepage rate, as predicted based on clear water and the native soil permeabilities, is, therefore, likely to decrease once the lagoon cells are placed into operation. The sealing ability of the sludge blanket in the bottom of lagoon cells appears to be most significant with more permeable soils. With impermeable soils, the sealing action may be insignificant. Reference [65] discusses U.S. experience with lagoon liners and natural sealing of lagoons.

In circumstances where it can be demonstrated that there will be no adverse impact on the environment, lagoons designed to allow the effluent to infiltrate into the ground will be

considered for approval. These exfiltration lagoon proposals will require hydrogeological studies to assess their potential impact on the environment.

19.6 DESIGN CAPACITY OF LAGOONS

In Ontario, it has been accepted practice to calculate the design capacity of lagoons based upon average daily sewage flow rates only, making no special allowance for net precipitation entering the cells. With the normal amounts of leakage occurring from lagoons and with the design freeboard allowances of at least 0.6 m, this design practice has proven to be acceptable.

19.7 DEEP LAGOONS

Non-aerated lagoon cells deeper than the normal 1.8 m operating depth will be permitted in either of the two following circumstances:

a) regardless of detention time in the cells, provided that the equivalent of secondary treatment is provided prior to the introduction of the sewage into the cells (influent BOD_5 and suspended solids should be in the order of 30 mg/L); cell operating depth of 2.7 m will be acceptable and greater depths will be considered on a case-by-case basis;

b) when a detention time of 12 months is provided, with discharge only allowed in the fall, with a minimum of two cells provided, and with a maximum organic loading rate of

11 kg/ha.d (except during the period of discharge); maximum cell operated depth of 2.4 m will be permitted.

The criteria for deeper cells in the case of 12 month retention lagoons is considered as interim criteria until the Ministry gains experience with the operation of these deep lagoon systems.

SECTION 20

20.0 AERATED LAGOONS

The most commonly used type of aerated lagoon in Ontario is the aerobic-facultative aerated lagoon. With aerobic-facultative aerated lagoons, enough oxygen is transferred to satisfy the applied BOD_5 loading and maintain an adequate dissolved oxygen level and the cell contents are mixed sufficiently to maintain uniform dissolved oxygen levels throughout the cell. No attempt is made, however, to supply enough mixing to maintain a uniform solids concentration in the aeration cell. In fact, mixing is kept low enough to permit solids settling in the cell. Solids settling to the cell bottom undergo anaerobic decomposition and the products of this decomposition are released and treated in the upper aerobic layers.

Completely-mixed aerobic lagoons are another type of aerated lagoon. With these lagoons, complete mixing is achieved and, in effect, the lagoon and aeration system are designed in a similar manner to aeration tanks used with mechanical sewage treatment plants, except that earthen berm construction is used. To justify the extra power for complete mixing, these systems must be followed by a sedimentation basin with sludge return to the aerated cell. See Section 10.0 for a discussion of the design of aeration basins.

Table 10.1 shows the organic loading rate, typical oxygen demand and dissolved oxygen levels which must be satisfied for aerobic-facultative aerated lagoon systems. Table 10.2 gives the mixing requirements to achieve uniform dissolved oxygen levels.

The most common application for aerobic-facultative aerated lagoons is for pre-treatment of raw sewage prior to discharge into waste stabilization ponds. With 4 to 5 days retention time, typical effluent quality for aerated lagoon systems treating domestic sewage will be a BOD_5 of 60 mg/L, SS of 100 mg/L, and P of 6.0 mg/L. With a total retention time of approximately 30 days, including the time in a quiescent zone to permit solids settling, effluent quality equivalent to that produced by conventional activated sludge plants can be achieved.

Various types of aeration systems may be used, including bridge-mounted mechanical surface aerators; floating mechanical surface aerators; diffused aeration, using diffusers or aeration tubing; etc. Where extreme winter temperatures are experienced, submerged aeration systems are recommended, but where mechanical surface aerators are used; they should be of the low-speed bridge-mounted type to avoid icing damage.

Erosion protection will generally be required below mechanical aerators to prevent bottom scour. Berm construction should be as outlined for waste stabilization ponds (Section 19.4) except that liquid depths of 3 to 4.5 m will generally be designed for. Inside slopes should be 4:1, unless a soils consultant's report indicates that steeper slopes may be used. A soils consultant's recommendation should also be obtained on the need for and type of liner and/or berm erosion protection necessary. The following factors should be considered when selecting the method of erosion protection:

- type of aeration system and turbulence levels produced;
- size of aeration cell;
- wind direction;
- soil type and susceptibility to erosion;
- berm slopes;
- liner material, if any;
- location of piping entrances and exists and chambers.

For the recommended land use surrounding lagoons see Appendix B. For other design considerations, such as soil and groundwater conditions, site drainage, fencing, etc. refer to Section 19.

SECTION 21

21.0 LAND APPLICATION OF TREATED SEWAGE EFFLUENT

The Ministry of the Environment has prepared draft policies[1] and guidelines relating to land application of treated sewage effluent to both agricultural and recreational lands.

Land application of treated sewage effluent is a method of disposing of effluent without direct discharge to surface waters. Effluent irrigation is a disposal alternative that can be used when there is insufficient assimilative capacity in nearby watercourses, or where downstream water uses will preclude direct effluent discharges to watercourses. Effluent irrigation can also be used as an alternative to advanced waste treatment with effluent discharge to surface watercourses. Other disposal methods, such as sub-surface disposal via tile fields, or pipeline conveyance to more acceptable receiving streams should also be considered as alternatives to land application of effluent.

Land application of treated sewage effluent takes advantage of the soil's and vegetation's capacities to renovate effluent by the combined processes of filtration, absorption, chemical

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1. Designers are advised that the treatment requirements for sewage prior to land application, as well as other policy matters, may change when these policies are finalized. Before a design is initiated, therefore, the status of these policies and their requirements should be checked.

precipitation, ion exchange, biochemical transformation and/or biological adsorption. There are a number of land application techniques, including spray irrigation, rapid infiltration basins, ridge and furrow systems and overland runoff systems. Spray irrigation has been the most widely used method of land application in Ontario. The following discussion will deal in general with land application of treated sewage effluent and will give some design details and design procedures for spray irrigation systems, in particular. A more detailed discussion of the other forms of land application can be found in references [19], [55], [56], [57], [58], and [59].

For successful operation, an effluent irrigation system will require:

- suitable soils;
- suitable topography and hydrological conditions;
- adequate site area at reasonable cost;
- suitable site isolation from conflicting land uses;
- suitable climate;
- effective site preparation;
- proper crop selection;
- good management;
- adequate waste treatment prior to irrigation;
- adequate effluent holding capacity for non-irrigation periods.

21.1 SOILS

A soils report must be submitted along with the application for approval of the effluent irrigation system. This report must not only demonstrate the suitability of the soils for waste stabilization pond construction which is normally required with land application systems, but also the acceptability of the soils' infiltration capacity and permeability to accommodate the proposed sewage effluent application rates.

The infiltration capacity refers to the rate at which water can enter the soil. If the application rate exceeds this capacity, surface ponding, runoff and erosion will occur, leading to deterioration in soil structure and a further decrease in infiltration capacity. This effect is particularly important with soils containing silt and clay.

Permeability refers to the ability of the soil to allow water to move through the soil. Permeability will generally vary with depth, with the surface soils generally being more permeable than subsoils. Soils testing must, therefore, determine the limiting permeabilities of the soils at the proposed site.

Other factors which the soils report should establish are the soil type and drainage characteristics, the soil strata, and the expected depth to the water table during the irrigation season.

21.2 TOPOGRAPHICAL AND HYDROLOGICAL CONDITIONS

A contour plan, showing contours not exceeding one-half metre intervals, should be prepared for the treatment and irrigation areas. The present and future directions of surface drainage and groundwater movement from the spray irrigation site should be determined and shown on the topographical plan. Depending upon the runoff coefficient for the site, the distance from watercourses and the degree of effluent renovation required, irrigation in areas close to watercourses may have to be prohibited. The potential for affecting nearby water supply wells should also be assessed.

Spray irrigation sites ideally should be as flat as possible to facilitate agricultural activities and to minimize runoff. Slopes on cultivated fields should be limited to 4 per cent. On grassland, slopes of 8 per cent may be acceptable. Steeper slopes on forested land may be acceptable, depending upon the period of spray irrigation. Depressions or ruts within spray areas should be filled or avoided to prevent stagnation or channelization of the effluent.

Consideration should be given to the need for emergency discharges of effluent from the treatment facilities. The method by which such discharges could take place and the route such discharges would follow to a watercourse should be defined.

The depth to the watertable in the irrigation area should be at least 2 m, unless the site is underdrained, in which case a drain depth of one metre is satisfactory.

Sites with runoff coefficients up to 0.35 are suitable for spray irrigation provided the unsaturated soil zone criteria are met. Soil permeabilities in the moderate to rapid class (10^{-4} to 10^{-2} cm/s) are generally considered ideal for spray irrigation systems. The spray area should be designed in such a way that surface runoff does not enter or leave the spray area.

21.3 SITE AREA REQUIREMENTS

Various factors will influence the size requirements for the irrigation area, including the following:

a) length of irrigation season;
b) volume of sewage effluent to be applied;
c) acceptable average application rate over the irrigation season.

For infiltration-percolation systems, the frost free period is the recommended limit for the length of the irrigation season when the land is not underdrained. When the land is underdrained, the mean annual growing season is the recommended limit for the length of the irrigation season for infiltration-percolation and overland runoff systems. For irrigation systems relying primarily upon evapotranspiration (minimum infiltration and runoff), the limit of the irrigation season will be the frost free period.

In Ontario, the mean frost free period ranges from a high of 172 days in the climatic region of Leamington to 75 days in the climatic region of Patricia. For the same regions, the growing seasons are 221 and 131 days, respectively. Table 21.1 which has been extracted from reference [55] shows the mean annual frost-free period, mean annual growing season and mean annual potential evapotranspiration for the climatic regions of Ontario. Figures 21.1 and 21.2 from the same reference show the climatic regions of Ontario and the mean May to September precipitation for Ontario.

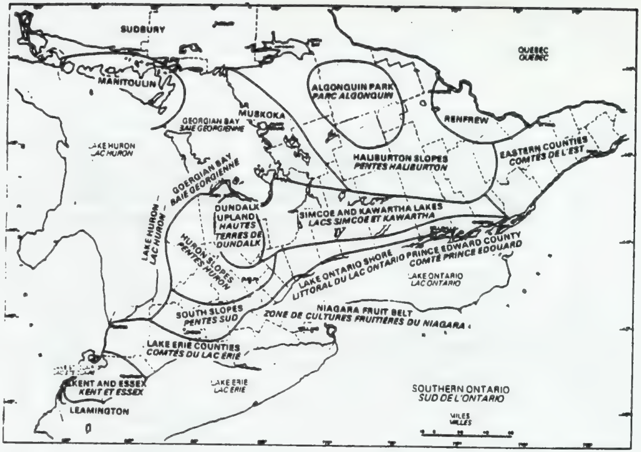
The amount of sewage effluent which may be applied by spray irrigation over a season will depend upon the infiltration/permeability of the soil and the crop water deficit. The crop water deficit is the sum of the potential evapotranspiration and the soil moisture-holding capacity minus the May to September precipitation. The crop water deficit is very small and usually amounts to only a few centimetres of liquid per year.

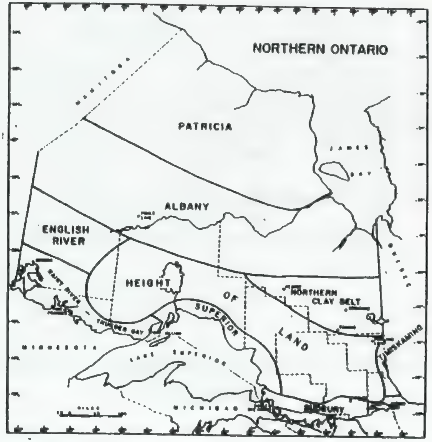
Regardless of the length of the frost-free period and the calculated average seasonal effluent application rate, the spray irrigation site area cannot be based upon a spray season in excess of 100 days nor upon an average effluent application rate in excess of 55000 L/ha.d.

21.4 SITE BUFFER ZONES

From the outside limits of waste stabilization ponds to dwellings, an isolation distance of at least 100 m should be provided (See Appendix B).

FIGURE 21.1
CLIMATIC REGIONS OF ONTARIO

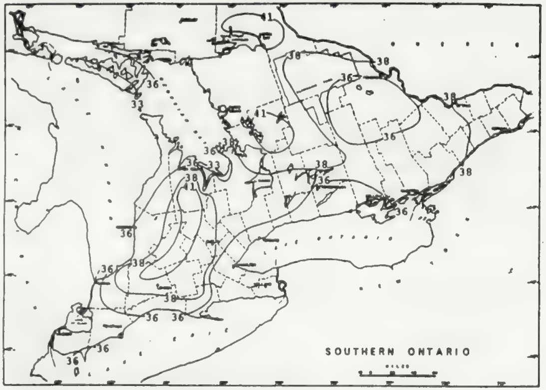


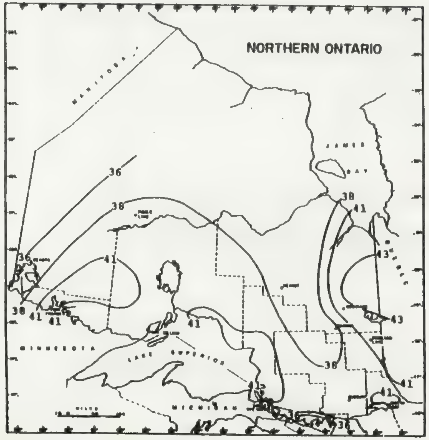


NOTE:

This information is from the Climate of Southern Ontario, Climatological Studies Number 5, Environment Canada Atmospheric Environment Service, and from the Climate of Northern Ontario, Climatological Studies Number 6, Department of Transport, Meteorological Branch.

FIGURE 21.2
MEAN MAY TO SEPTEMBER PRECIPITATION (cm)





This information is from the climate of Southern Ontario, Climatological Studies Number 5, Environment Canada Atmospheric Environment Service, and from the Climate of Northern Ontario, Climatological Studies Number 6, Department of Transport, Meteorological Branch.

For larger lagoon facilities, distances up to 400 m may be required to minimize odour problems.

In the absence of detailed assessments, the distance from spray nozzles to the property limit shall be 150 metres. Spraying is possible at closer distances from the property limit provided that low pressure, low angle, closely spaced sprinklers are used to minimize the formation of aerosols. In addition, the risk associated with aerosols can be minimized by providing a fence or tree screen around the site perimeter and by terminating spraying operations when wind speeds exceed that of a gentle breeze (15 km/h). Effluent disinfection may be required where the above measures do not provide sufficient protection.

Lagoon and irrigation areas should be enclosed with suitable fencing to exclude livestock and discourage trespassing (see also Section 19.4). Vehicle access gates should be provided where necessary to accommodate maintenance and supply vehicles and agricultural equipment. All access gates should be locked. The perimeter fences and gates should be provided with appropriate signs designating the nature of the facility and prohibiting trespassing.

21.5 PILOT TESTING

On-site pilot testing is recommended to determine the feasibility of land application of treated sewage effluent and to provide design data on application rates and amounts. Reference [55] covers the procedures to be followed with pilot testing.

21.6 PRE-TREATMENT REQUIREMENTS AND CROP SELECTION

Treated sewage effluent cannot be irrigated on crops used for direct human consumption. Land which has been previously irrigated with secondary effluent, or equivalent, can be used for such crops however, provided that a period of at least 6 months has elapsed since the last effluent application.

With crops used for animal consumption, land application of waste stabilization pond effluent or normally disinfected (chlorination at 0.5 mg/L residual and 30 minute contact time) secondary effluent from a mechanical plant is permitted, but for dairy cattle pastures the waste must have received the equivalent of secondary treatment plus disinfection to the bacteriological criteria for swimming and bathing use of water (geometric mean densities of less than 100 and 1000 per 100 mL for fecal and total coliforms, respectively). Treatment provided by a waste stabilization pond, designed to the standards outlined in Section 19.4 with at least 30 days retention time since the last addition of raw sewage prior to spraying is considered equivalent to secondary treatment and may achieve the above-mentioned bacteriological criteria without disinfection being required.

If the land is not to be used for at least one-half year after spraying, disinfection will not be necessary. For other pasture, silage, haylage, orchards, etc., the effluent must be normally disinfected (chlorination to 0.5 mg/L residual and 30 minutes contact time) secondary effluent must be normally disinfected

(chlorination to 0.5 mg/L residual and 30 minutes contact time) secondary effluent or waste stabilization pond effluent without disinfection. With orchards, non-spray application methods must be used, (e.g. ridge and furrow or gated pipe). In all cases, the crop should be allowed to dry before harvesting or pasturing.

With recreation lands, the treatment requirement is secondary biological activated sludge treatment, or equivalent, with the resulting effluent being discharged to the first two ponds connected in series, each with a retention period of not less than 30 days. The effluent to be sprayed must have a geometric mean fecal coliform MPN level of 14 per 100 mL, or less, based on a minimum of 10 samples over a 30 day period. Disinfection will generally be required to achieve this bacteriological quality. A free chlorine residual of 0.5 mg/L, or more, in the effluent at the end of a 30 minute contact time shall be deemed to have achieved this criterion (sampling and bacteriological analyses will not be necessary if such a free chlorine residual is present).

On arable land, perennial grasses (Brome, Orchard, Reed Canary and Timothy) are most suitable for spray irrigation disposal sites as they have fibrous root systems; are sod forming, which aids in erosion control and provides for high infiltration rates; have a long period of growth; and have a high uptake of nutrients [60]. The order of preference for use of these grasses on disposal sites is as follows:

Order	Name	Comments
1	Reed Canary	- Tolerates excessive moisture and is highly productive for long term hay or pasture on poorly drained soils, or areas subject to prolonged periods of flooding; less palatable than other grasses; more acceptable to livestock when stored as silage or haylage rather than dry hay.
2	Timothy	- Well adapted to heavier soil types and variably drained soils.
3	Brome	- Highest digestibility of grasses when cut at the "heads emerged" stage; superior for early pasture; good growth in fall.
4	Orchard	- Requires well-drained sites to avoid winter kill; grows back immediately after cutting or grazing.

TABLE 21.1

CLIMATIC SUMMARY FOR ONTARIO (1)

LOCATION	MEAN ANNUAL FROST-FREE PERIOD		MEAN ANNUAL GROWING SEASON		MEAN ANNUAL POTENTIAL EVAPOTRANSPIRATION
	DATES	DAYS	DATES	DAYS	(cm)
Leamington	May 1-Oct.20	172	Apr.8-Nov.12	221	66
Niagara Fruit Belt	May 5-Oct.15	163	Apr.10-Nov.10	215	64
Kent and Essex	May 5-Oct.15	163	Apr.8-Nov.11	218	66
Lake Erie Counties	May 12-Oct.10	151	Apr.10-Nov.8	213	64
Lake Ontario Shore	May 12-Oct.8	149	Apr.12-Nov.3	206	61
Prince Edward County	May 12-Oct.10	151	Apr.12-Nov.5	208	61
Lake Huron, Georgian Bay	May 15-Oct.10	148	Apr.15-Nov.5	205	61
South Slopes	May 15-Oct.5	143	Apr.13-Nov.3	205	61
Huron Slopes	May 20-Sept.30	133	Apr.17-Oct.31	198	58
Simcoe and Kawartha	May 18-Sept.28	133	Apr.18-Oct.28	194	58
Eastern Counties	May 15-Sept.28	136	Apr.15-Oct.28	197	61
Manitoulin	May 25-Sept.28	126	Apr.23-Oct.28	189	56
Muskoka	May 25-Sept.25	123	Apr.22-Oct.27	189	56
Renfrew	May 18-Sept.25	130	Apr.18-Oct.27	193	58
Dundalk Upland	May 31-Sept.20	113	Apr.20-Oct.25	189	56
Haliburton Slopes	May 25-Sept.17	115	Apr.22-Oct.24	186	56
Algonquin Park	May 31-Sept.20	113	Apr.25-Oct.21	180	53
Sudbury	May 31-Sept.20	112	Apr.25-Oct.24	183	56
Thunder Bay	May 31-Sept.12	104	Apr.26-Oct.17	175	53
Timiskiming	June 10-Sept.13	95	Apr.27-Oct.15	172	53
Superior	June 5-Sept.15	103	May 6-Oct.15	163	51
Northern Clay Belt	June 8-Sept.7	92	May 7-Oct.13	160	51
English River	May 30-Sept.15	108	May 3-Oct.13	164	53
Height of Land	June 15-Sept.2	80	May 5-Oct.13	162	46
Albany	June 12-Sept.5	86	May 15-Oct.8	154	46
Patricia	June 18-Aug.31	75	May 24-Oct.1	131	41

NOTES :

1. This Climatic Summary for Ontario is from Climatological Studies Nos. 5 and 6, Canada Dept. of Transport, Met. Branch.

TABLE 21.2

MOISTURE REQUIREMENTS FOR PERENNIAL GRASSES (1.)

SOIL	APPLICATION AMOUNT (cm)	PERIOD BETWEEN IRRIGATION APPLICATIONS (days)	RECOMMENDED APPLICATION RATE (cm/h)
Well Drained Sands	3.3	5	0.6-1.9
Loamy Sands	4.3	6	0.6-1.3
Light Coloured Loams and Sandy Loams and Good Drainage	5.1	7	0.6-1.3
Dark Doloured Loams and Sandy Loams with Fair to Poor Drainage	6.9	10	0.6-1.3
Clay Loams	6.1	9	0.4-1.0

NOTE :

1. This information is from Irrigation Practices for Ontario, OMAF AGDEX 560/753

Forests and brushland should also be considered for wastewater disposal as the land value is relatively low compared to cultivated areas.

21.7 MOISTURE REQUIREMENTS AND EFFLUENT APPLICATION AMOUNTS

The moisture requirements for perennial grasses grown in various soil types are shown in Table 21.2. The application amount should be applied over a one-day period and should not be repeated until the number of days specified in the period between irrigation applications has passed. The cycle is then repeated throughout the irrigation season.

The application amount is the quantity which should be needed to maintain the soil at "Field Capacity". Any combination of rainfall or effluent quantities above this amount will percolate through the soil to the watertable. If the sum of the effluent application and rainfall is equal to or less than the "application amount", maximum plant utilization of the effluent nutrients and minimum infiltration to the groundwater should occur.

The effluent application amounts for infiltration and evapotranspiration spray irrigation systems should be as follows:

a) For infiltration systems, the following restrictions must all be taken into account for a system to operate properly.

   - The hourly effluent application rate must be less than the surface infiltration rate measured in cm/h.

- the daily effluent application rate plus the week's rainfall on an area of the spray field that is irrigated one day per week measured in cm/d must be less than the permeability of the most impermeable soil sub-horizon measured in cm/wk.

- For optimum utilization of nitrogen in the effluent, the following equation applies:

$$W = \frac{10C}{Y-A}$$

  Where:
  W = annual wastewater loading in cm/a
  C = removal of nitrogen by crop in kg/ha.a (uptake by Reed Canary Grass = 335 kg/ha.a)
  Y = total nitrogen concentration in effluent in mg/L
  A = allowable nitrogen concentration in percolate in mg/L (Drinking Water Limit = 10 mg/L NO_3 as N)

b) For minimum infiltration runoff, i.e. maximum evapotranspiration:

determine mean annual frost free period (Table 21.1), divide by irrigation period (Table 21.2), multiply by application amount (Table 21.2) and subtract mean May to September precipitation (Figure 21.2); the application rate (Table 21.2) which is used should be the minimum.

21.8 SPRAY SYSTEM DESIGN FEATURES

Spray irrigation areas should be divided into sections so that the number of sections equals the number of days in the irrigation period. Spraying can then be carried out on a rotation basis with the most effective use being made of the spray area and equipment.

Irrigation piping in the spray areas should allow for flexible operation, including selection of spray areas, isolation of piping sections, etc. Although stationary piping systems may be used for small systems. Traveling sprinklers may also be considered provided the topography is suitable. Stationary piping systems must be designed to permit drainage to prevent freezing damage. Valves, sprinkler heads, and pipelines should be colour coded and designated as carrying treated sewage, to prevent cross connections and improper use.

Sprinklers must be provided in such a pattern that full field coverage is achieved. Some overlapping of the spray patterns will be necessary to ensure total coverage. Unirrigated areas would result in excessive weed growths in the drier areas with lower crop values.

A flow meter should be provided to permit measurement of the application rates and amounts. A pressure gauge should also be provided to monitor line and sprinkler head losses.

21.9 SITE CONTROL

The proponent of irrigation systems must be able to demonstrate that the irrigation lands will be available when needed to dispose of sewage effluent. This will normally mean that the lands must be owned by the proponent.

Non-ownership of the irrigation lands will be considered, however, provided that they are leased over a long enough term and with renewal clauses to satisfy the Ministry of the Environment that alternate disposal options could be developed, if found necessary, and provided that the terms of the lease grant the owner of the sewage treatment system the right to irrigate even if such action may destroy or damage the crops being grown. This latter provision will be necessary to ensure that the satisfactory disposal of sewage effluent will take priority over any cropping activities when, and if, found necessary. The lease should, therefore, include terms to compensate the land owner for crop damage, or loss, in such eventualities.

SECTION 22

22.0 PLANT PIPING

All piping to be used in sewage treatment plants should be manufactured in accordance with CSA, CGSB, ASTM, or other internationally recognized standards. Digester gas, propane, fuel oil and natural gas piping must further comply with the requirements of B 105 [43], as was previously discussed in the subsection on Anaerobic Digestion.

In the design of the piping, due allowance should be made for future capacities and also the ease of extending this piping without major disturbance of the plant. In the general piping arrangement, sufficient space should be provided for piping to be removed, and the pipe design should provide for the proper isolation through valves, of pipe sections to enable them to be repaired or replaced.

In larger plants, galleries are often used for the location of process piping and for the passage of operating staff between buildings and tankage units. Tunnels may be formed by using the walls of adjacent tank structures and the floor slab may be common to all structures. Although galleries will generally cost more than buried piping systems, their use may be justified due to the more convenient plant operation and access to process piping for maintenance.

The designer should allow for the possibility that piping could be installed during construction when temperature conditions could be substantially different from the design condition (for example, piping could be installed in temperatures anywhere between +40°C and -20°C),

and substantial differences in pipe lengths could occur. For this reason the use of PVC pipe with cast iron mechanical joint fittings is not recommended. Where piping is cast-in-place, due allowance should be made for differential expansion between the pipe material and structures.

Piping should be arranged so that all valves, flow meters, and other items which may require regular inspection or maintenance are conveniently accessible. Piping should be provided with drains at all low points, and air release valves at all high points. Sludge and scum piping should be provided with cleanouts and facilities to permit water and/or steam cleaning. Scum piping should be smooth-walled pipe, preferably glass lined.

The design of the piping should allow for proper restraint under all anticipated conditions, particularly where surges may occur and high transient pressures could result, or where different temperatures occur seasonally.

Where piping connections are made between adjacent structures, at least one flexible coupling should be provided if there is any possibility that differential settlement could occur. Particular attention should be given to pipe bedding in areas adjacent to structures to avoid settlement damage.

Where a sewage treatment works obtains water from a municipal potable water supply, the municipal water supply must be protected with an approved backflow preventer at each point of

connection with the municipal system. In addition, at each potential point of contamination of the water supply within the sewage treatment works, an approved backflow preventer must be installed.

Under The Occupational Health and Safety Act, piping identification, as to flow direction and contents, is mandatory only for piping systems containing hazardous substances. However, it is recommended that all piping be adequately identified as to contents and direction of flow so that the operation of the process units is simplified. Piping identification is recommended to be by complete painting of the pipe line and by use of colour banding. Where there is no previously existing standard colour coding, it is suggested that the following code as outlined in Table 22.1 be used for pipe colour.

Clearly visible lettering to indicate the actual pipe contents, e.g. raw sludge, waste activated sludge, etc., should be shown on colour bands along with the flow direction arrow. To comply with CSA B53, the bands should be coloured as shown in Table 22.2.

TABLE 22.1

WATER POLLUTION CONTROL PLANT PIPING AND STORAGE VESSEL COLOUR CODE

MATERIAL HANDLED	PIPE COLOUR	CGSB COLOUR CODE NO. (STD. 1-GP-12b)	CIL NO.
Sewage	Light Grey	501-108	9562
Sludge and Scum	Dark Brown	504-102	9566
Non-Potable or Effluent Water	Blush Yellow	504-107	2916-8
Potable Water	Light Blue	502-106	Special
Potable Hot Water	Dark Blue		955
Drainage	Black	512-101	95580
Chlorine Gas	Yellow	505-110	4724-5
Chlorine Solution	Yellow with Light Blue Band	505-110 502-106	4724-5 Special
Digester Gas, Natural Gas, Propane or Fuel Oil	Orange	508-102	4601-5
Compressed Air	Vine (Gloss)		4776-9
Circulating Air	Vine (Flat)		4776-9
Alum	Metallic Green	503-323	3624-8
Ferric chloride	Metallic Green with Orange Band	503-323 508-102	3624-8 4601-5
Polyelectrolytes	Metallic Green	503-323	3624-8
Sodium or Calcium Hypochlorite	Yellow with White Band	505-110 513-101	4724-5 9580
Lime	White with Orange Band	513-101 508-102	9580 4601-5
Ozone	Carmel	504-108	2885-8
Other Chemicals	Fanestia	511-104	4869-9

TABLE 22.2

WATER POLLUTION CONTROL PLANT
BAND COLOUR CODE

CLASSIFICATION	CLASSIFICATION COLOUR	CGSB	CIL
Dangerous Materials	Yellow	505-102	2007
Safe Materials	Green	503-107	94233
Protective Materials	Blue	202-101	95547
Fire Protection	Red	509-102	95557
Gas Piping, Controls and Flammable Gas	Orange	508-103	4601-5

For gas piping, CAN 1- B105 -M81 states that, "all gas piping and controls shall be painted or colour coded with high visibility paint and each system of piping shall be labeled every linear 3 m with the name of the gas being conducted and the direction of flow".

In sizing, and selecting the material and pressure requirements of piping for use in sewage treatment plant processes, the following factors must be considered:

- likelihood of blockage and size of line required;

- line size required to produce scouring velocities and thus minimize solids deposition and grease buildup;

- nature of material to be conveyed and suitable piping materials for the application;

- flow characteristics of material to be conveyed and head requirements of pumps, or differential head required for gravity flow;

- possible settlement and need for support;

- need for future repair;

- need for future removal of pipe sections.

The recommended minimum diameters of piping for various purposes is listed below:

<u>Gravity Flow</u>	<u>Minimum Diameter</u> (mm)
Waste water and sludges	200
<u>Pumped Flow</u>	
Waste water	100
Sludges	150
Chemicals (non-scale-forming)	12
Chemicals (scale-forming)	25

SECTION 23

23.0 INSTRUMENTATION AND CONTROL

23.1 GENERAL

The requirements for instrumentation and control will depend on the size of plant and types of process employed. In general, instrumentation and control should provide safe and efficient manual and automatic operation of all parts of the plant, with minimum operator effort, and all automatic controls should be provided with manual back-up systems.

Where some parts of the plant may be operated or controlled from a remote location, local control stations should be provided and shall include the provision for preventing operation of the equipment from the remote location. Consideration should be given to providing communication via intercom between remote stations and the local stations. In some cases, the use of television equipment may be justified to provide scanning functions of local instrumentation control centres as well as process equipment.

Decisions will have to be made by the designer as to which equipment will be controlled locally and which will be controlled from a remote location, and whether control will be automatic or manual. For instance, at a small plant, scum pumping may be controlled locally and manually, whereas raw sewage pumping should be automatically controlled, regardless of plant size. In addition, the points of control and the type of primary device must be selected. Decisions must

also be made as to whether the instrumentation is to totalize, indicate and/or record and whether alarm functions are to be incorporated.

In making the decisions relating to instrumentation and control, the following factors should be considered:

- plant size;
- effluent requirements;
- plant process complexity;
- hours in day plant will be manned;
- potential chemical and energy savings with automation;
- reliability of primary devices for parameter measurement;
- preferred location for primary device;
- parameters with useful significance to process;
- equipment which should be controlled automatically;
- equipment which should be controlled manually;
- equipment which should be remotely controlled;
- equipment which should be locally controlled;
- data requiring display at the control centre;
- indication, totalization and recording functions necessary to the overall process.

For proper operation of larger sewage treatment plants, the following parameters should be measured (however for smaller plants some of the parameters could be omitted):

- sewage flow rates, including raw sewage, by-passed flows, and flows through plant subsections (flow trains);
- chlorine dosage;
- sludge pumpage, including raw, digested sludges and activated sludge return;
- digester supernatant flows;
- chemical dosage with phosphorus removal processes;
- digester gas production and utilization;
- anaerobic digester temperature;
- hazardous gas levels.

Auxiliary instrumentation is desirable to measure the following parameters:

- air flow;
- chlorine residual;
- mixed liquor dissolved oxygen concentrations;
- sludge blanket levels;
- sludge concentrations.

23.2 TYPE OF INSTRUMENTS

The type of instruments that will be required to measure the parameters mentioned above are classified as prime element devices which transform a signal from the physical process to a suitable signal for transmission via a transmitter. These devices are broken down into various parameters and each of the parameters are further broken down into other parameters with a brief description of a particular process application.

23.2.1 Density Elements

The following points generally apply to density elements:

a) Radiation Type

i) Accuracy 0 to 15%

ii) Very difficult to keep in calibration; requires frequent maintenance.

iii) Requires certified personnel to handle radioactive material.

b) Ultrasonic

i) Range 1 to 10%

ii) Accuracy repeatability is $\pm 0.5\%$

iii) Avoid use where entrained gases are present

23.2.2 Dissolved Oxygen Elements

a) Volumetric Types

There are basically two types of volumetric meter available. They are active oxygen electrodes and passive oxygen electrodes.

23.2.3 Flow Elements

a) Magnetic Flow Meter

The magnetic flow meter operates on the principle that a conductor passing through a magnetic field will produce a DC voltage directly proportional to the speed of the conductor. The following points generally apply to magnetic flow meters:

i) Preferably installed with slight incline with upward flow.

ii) The meter must be full for proper operation.

iii) Should have a minimum velocity of approximately 2-4 m/s.

iv) Piping arrangement error is minimum and is not a substantial factor.

v) Avoid use where entrained gases are present.

b) Parshall Flume

i) Flume must be level.

ii) Accuracy affected by upstream channel arrangement (should have at least ten channel widths).

iii) Accuracy approximately 5% of rate.

c) Ultrasonic Flow Meter

The following points generally apply to ultrasonic flow meters:

i) No contact, with medium being measured.

ii) If doppler type, then the presence of entrained gases or solids are required for proper operation

iii) If grease is present in the medium, then heat or ultrasonic cleaning should be used.

iv) Accuracy $\pm$1-2% of scale.

d) Venturi Tube

The Venturi tube operates on the principle that the pressure differential between the inlet and the throat is proportional to the square of the flow.

The following points generally apply to Venturi tubes:

i) Meter must be full for proper operation.

ii) Accuracy is affected by upstream piping arrangement.

iii) If used on sludge of any kind, then water purge arrangement should be used.

iv) Accuracy ±2% of rate; however, accuracy will be much lower at the low end, depending on which type of transmitter is used.

v) If time differential type, then the present of entrained gases and/or solids will affect the accuracy severely.

vi) Accuracy affected by upstream and downstream piping arrangement.

vii) Accuracy ±2% of scale.

e) Volumetric Meter

The volumetric flowmeter measures flows in pumping stations controlled by low and high level contacts.

It does not require cleaning or recalibration as no parts are in contact with sewage and can be installed remotely from the station. The measuring principle of flowrate equals Volume divided by Time.

i) Programmed with on-site wet-well volumes.

ii) Digital display of inflow and outflow rate, total flow and pumping time.

iii) Accuracy ±0.1%.
CAUTION: Should not be used where variable speed pumping is employed.

f) <u>Rotameter</u>

The Rotameter is a tapered tube that has a ball float permitting rough visual readings. Mostly used in chlorinators and/or ammoniators where it is a standard fixture. Since it is installed in the flow line, it must be line sized and has limitations when transmission of a signal is required.

g) <u>Gas Meters</u>

a) Rotary positive displacement
b) Turbine

23.2.4 Level Elements

The following points generally apply to level elements:

a) <u>Bubbler</u>

i) Range 0 to 56 metres

ii) Accuracy $\pm$0.1% of actual reading

However, the accuracy will be further affected by the type of transmitter selected.

b) <u>Float Type</u>

i) Range of 0 to 11 metres

ii) Accuracy $\pm$1% of actual reading

c) Ultrasonic

i) No contact with medium being measured

ii) Accuracy ±2% of actual reading

d) Capacitance Probe

23.2.5 Pressure Elements

The following points generally apply to pressure elements:

a) Bellow (Lower Pressures)

i) Pressure range 10-2000 kPa

ii) Accuracy ±1% of actual reading

b) Bourdon Tube (Higher Pressures)

i) Pressure range 0-35 000 kPa

ii) Accuracy ±1% of full scale

c) Liquid to Air Diaphragm

(Commonly used in sensing pressures involving corrosive chemicals)

i) Pressure to approximately 20 m water column

ii) Accuracy ±1% of scale.

d) <u>Liquid to Liquid Diaphragm</u>

i) Pressures to approximately 20 m water column

ii) Accuracy ±1% of scale

e) <u>Strain Gauge</u> (Commonly used in conjunction with a bellow)

i) Should have temperature compensation

ii) Accuracy ±1% of reading

iii) Not sensitive enough for low pressure ranges

23.2.6 <u>Sludge Blanket Detector</u>

There are basically two types of sludge blanket level detectors available: one is the Photocell type and the other is the Ultrasonic type. If the application requires an ON/OFF type of control, then the Photocell type may be suitable. However, if an analog type of control or monitoring is required, then the Ultrasonic type will be required.

23.2.7 <u>Temperature Elements</u>

The following points generally apply to temperature elements:

a) <u>Filled System</u>

i) Gas filled - most common in sewage treatment plant application

ii) Temperature range 0-100°C

iii) Accuracy ±1% full scale

b) Resistance Temperature Detector

i) Use of thermowell advised

ii) Temperature range 0-100°C

iii) Accuracy ±0.5% of actual reading

c) Thermocouple

i) Use of thermowell advised

ii) Temperature range approximately 0-1000°C

iii) Accuracy ±1% full scale

23.3 PROCESS CONTROLS AND INSTRUMENTATION

23.3.1 Pumping Stations

The pumping station will require dependable and simple instrumentation and controls. The parameters that require monitoring and control in the pumping station are: level, flow, pumps and motors and alarms.

a) Level Control

The purpose of a level control is to regulate the pumping rate of waste water to the treatment plant. If the pumps are variable speed an analog monitoring and

control system should be used. If the pumps are constant or multiple speed a stepping type of control system should be used.

b) <u>Flow Monitoring</u>

The flow metering device should be selected very carefully to ensure that there are no obstructions where clogging may occur. Routine preventative maintenance should also be considered in the selection of the flow meter.

c) <u>Pumps and Motors</u>

The following parameters should be monitored:

i) <u>Pump</u>

- Bearing temperature
- Casing temperature
- Vibration
- Speed
- Suction and discharge pressure

ii) <u>Motors</u>

- Voltage
- Current
- Hours of operation
- Bearing temperature
- Windings temperature

d) Alarms

Alarms are the last and final warning to the operator that the system is malfunctioning and unless corrective action is taken, damage may occur. Audio/visual alarms are recommended in order to focus operator's attention on the actual fault condition.

A pumping station should have at least the high and low liquid level alarm and other pump alarms, including motor winding temperature, pump and motor bearing temperature, and motor overload.

23.3.2 Mechanical Bar Screens

There are two methods to control the mechanical bar screens:

a) Simple manual start/stop which requires the presence of an operator at the screen in order to start and stop.

b) Automatic which operates when activated by a differential pressure switch. The screen should run for at least one complete screen revolution. To achieve the minimum running time a timer or limit switch may be used. In addition a timer should be provided to ensure periodic cleaning of the screen, regardless of actual headloss.

c) Alarms

There should be a differential pressure switch for alarm signal with a head loss setting of approximately 100 mm higher than the setting for automatic start-up of the mechanical bar screen.

23.3.3 Primary Treatment

a) Raw Sludge Pumping

The raw sludge pumps should be set up in such a way that the following features are incorporated:

- selectable pump duty (manual selection of duty pump)
- manual override of automatic controls
- individually selected hopper pumping times
- adjustable density control
- sludge flow
- sludge density

b) Scum Pumping

The scum pumps should be set up in such a way that the following features are incorporated:

- selectable pump duty
- manual override of automatic controls
- automatic control system consisting of scum high and low level switches
- scum temperature indicator

The duty pump should start at scum high level and stop at scum low level in the scum tank.

23.3.4 Secondary Treatment

Automatic D.O. (dissolved oxygen) control systems can be used to control the rate of air supply to the aeration tanks. The use of D.O. control can result in energy and money savings. There are several different methods of automatic D.O. control and the most used are:

a) Flow Ratio

This consists of a fixed ratio of volume of air to plant influent flow.

b) Closed Loop Control

This consists of D.O. probes and controllers where the actual D.O. reading is compared to a set point on the controller and the resultant error signal is used to increase or decrease the oxygen supply to the aeration tanks.

There are other methods, such as F/Mv ratio or MCRT (mean cell residence time) control; however, these methods require the use of a computer for calculation, forecasting, and modeling.

23.3.5 Chemical Control Systems

The chemical additions with the exception of the chlorine system is a feed forward control system. This consists of a feeder or chemical

metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control.

The chlorine addition is a compound loop control system which consists of adjustable constant ratio of chlorine to influent or effluent flow with trim based on chlorine residual as measured by a chlorine analyzer.

The chlorine addition for larger treatment plants should consist of at least three chlorinators and two analyzers:

- one chlorinator for pre-chlorination
- one chlorinator for post-chlorination
- one chlorinator standby that can be used as either pre or post-chlorination
- one analyzer for post-chlorination
- one analyzer for pre-chlorination

Each analyzer should be capable of being switched to the standby chlorinator.

23.4 CONTROL AND MONITORING SYSTEMS

There are two control and monitoring systems available. One is the conventional system with recorders, indicators, switches, push buttons, indicating lights, control panels etc., and the other is the computerized system.

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions.

The computerized system is a multi-purpose system with limited scope for modification or a dedicated purpose system with standard hardware and customized software.

Both computerized systems have two basic configurations:

i) A centralized configuration where all intelligence is resident in the computers in Central Control Station.

ii) A distributed control configuration where the intelligence is distributed throughout the system.

The distributed system hardware costs will be higher than the centralized configuration; however, wiring and installation costs will be less. There are several important advantages to the distributed system.

One of the advantages is that with intelligence distributed throughout the system, the software required for the computers at Central Control becomes less complicated and requires less maintenance. The intelligence contained in the other components of the system will be on firmware which requires no maintenance. Another advantage is that in the event of a communication failure, each intelligent component in the system can operate on its own and maintain some pre-programmed condition based on its own sensors. Therefore, when a lower intelligence component loses communication with a higher

intelligence component, it will still function with the pre-determined fail-safe program to maintain system operation.

The computerized systems can be arranged so that all operating decisions can be made by the computer based on instructions given at an earlier stage of the formal programming. Alternatively, the terminal equipment can be used for information display and manual initiation of control commands, that is as a remote manual control station.

23.5 SECURITY

The security systems covered in this section refer specifically to electronic/electric type surveillance and intruder alert systems. Fencing and other safety aspects are covered elsewhere.

In larger sewage treatment plants, the main gate should have at least one of the following access control systems.

a) Punched or magnetic cards with a card reader at a central control station.

b) Closed circuit TV system where the operator has to operate the gate/door from a remote location.

c) An intercom system where operator has to operate the gate/door from a remote location.

In remote pumping stations, and if required in in-plant buildings, there should be legal/ illegal entry alarm systems. These systems should include door and window switches, tapes, etc. that will provide an indication to a central location that a legal or illegal entry has been made.

SECTION 24

24.0 LABORATORY FACILITIES

MOE Policy 08-06 and its related guidelines cover the sampling and analysis requirements for sewage treatment plants. MOE Regional Staff should be consulted to determine the requirements for any particular sewage treatment plant proposal.

In general, adequate space and equipment should be provided so that all necessary testing may be done on site which is essential for proper treatment process operation and control. The following equipment is recommended as minimum requirements for sewage treatment plants (item 4 only mandatory if nitrification is a treatment requirement):

1. Drying oven and muffle furnace for suspended and volatile solids analyses;

2. Field type dissolved oxygen probe and meter;

3. Incubator and associated apparatus for doing BOD analyses;

4. Kits and chemicals for ammonia, nitrate and nitrite analyses;

5. Kit for soluble phosphorus analyses;

6. Microscope having the capability of up to X400 magnification with an electrical light source;

7. Glassware associated with the above tests, including 1 litre graduate cylinders for solids settling measurements;

8. pH meter and accessories;

9. Balance;

10. Amperometric chlorine residual analysis equipment.

For smaller plants where it can be demonstrated that analytical work can be carried out and results obtained within a reasonable time frame through commercial, government laboratories, or other approved agencies, some of the above equipment may be omitted from the plant laboratory.

The minimum linear space recommended for laboratory facilities is 3 m, including a wash-up sink.

In large plants, it is recommended that sample lines be brought to the laboratory from the various stages of the treatment process, including raw sewage, primary effluent, final effluent, chlorinated final effluent, etc. These sample lines should run continuously to provide representative samples of the various waste streams. The sample lines should be kept as short as possible. Where sample lines are not provided, convenient sampling points for the automatic or manual collection of samples should be provided at all pertinent locations in the treatment process.

SECTION 25

25.0 SAFETY

The designer is referred to the "Safety Manual" [1] prepared by the Ministry of the Environment, the Occupational Health and Safety Act, and Regulations for Construction Projects [205]; the Occupational Health and Safety Act, and Regulations for Industrial Establishments [206]; the National Fire Code of Canada (NRCC Publication 14987) [204]; the booklet "Hazardous Chemicals Data 1975" (NFPA 49) [41]; and CAN1-B.105-M81 "Installation Code for Digester Gas Systems", Canadian Gas Association [43].

Equipment suppliers and chemical suppliers should also be contacted regarding particular hazards of their products, and the appropriate steps taken in the facility design to ensure safe operation.

SECTION 26

26.0 PERSONNEL FACILITIES

The necessity for personnel facilities will be largely dictated by the number of operating staff required, and the time periods during which the plant is manned.

As a minimum it is recommended that provision be made for storage lockers, preferably two for each employee (one for work clothes, one for clean clothes), and a washroom with shower. As the size of the plant and number of staff increases, there will be a requirement to provide more locker space, possibly in a separate change room, a lunch room which should be of adequate size to serve as a meeting or instruction room for plant staff, and a suitable office for plant supervisory staff and record keeping.

Whenever possible, these personnel facilities should be separated from the plant facilities, but with convenient access to the plant.

SECTION 27

27.0 BUILDING SERVICES

Adequate heating facilities of a safe type should be provided with control levels depending on the type of area being heated. In many areas of the plant, sufficient heat need only be provided to prevent freezing of equipment or treatment processes.

Buildings should be well ventilated by means of windows, doors, roof ventilators, or other means. All rooms, compartments, pits, and other enclosures below grade which must be entered should have adequate forced ventilation provided when it is necessary to enter them.

Rooms containing equipment or piping should be adequately heated, ventilated, and dehumidified, if necessary, to prevent undue condensation. Switches should be provided which would conveniently control the forced ventilation.

Buildings should be adequately lighted throughout by means of natural light, artificial lighting facilities, or both. Control switches where needed should be conveniently placed at each entrance to each room or area.

As discussed in the previous section "Instrumentation and Control", it may be advantageous to provide intercom systems between the control centre and other buildings or locations throughout the plant site. In certain circumstances television monitoring may also be warranted. Public telephone service should at

least be provided to the control centre and other manned centres throughout plant. Empty conduit systems may also be provided for future telephone and/or intercom lines.

Power outlets of suitable voltage should be provided at convenient spacing through plant buildings to provide power for maintenance equipment, extension lighting, etc. Power outlets should also be located at outside locations to permit servicing of such equipment as scraper drive mechanisms, flow meters, comminutors, etc.

Potable water service will also be required to most buildings. Reference should be made to Section 22 for requirements relating to back-flow prevention for potable water supplies. Wherever possible, to conserve energy and minimize operating costs, effluent water should be used for water uses not requiring potable water. Such uses as chlorinator - injector water, lawn sprinkling, foam control, flushing water, filter press belt wash, and incinerator off-gas scrubber water can be supplied by plant final effluent. Fixtures supplied with non-potable water should be clearly marked as supplying non-potable water.

APPENDICES

APPENDIX 'A'

GLOSSARY OF SYMBOLS AND ABBREVIATIONS

APPENDIX 'A'

GLOSSARY SYMBOLS AND ABBREVIATIONS

a	- year (annum)
$Al_2(SO_4)_3 \cdot 14H_2O$	- aluminum sulphate
BOD	- biochemical oxygen demand
BOD_5	- 5-day biochemical oxygen demand
Soluble BOD_5	- soluble biochemical oxygen demand
°C	- degree Celsius
$CaCO_3$	- calcium carbonate
$Ca(OH)_2$	- calcium hydroxide
d	- day
D.O.	- dissolved oxygen
EP Act	- Environmental Protection Act
EPA	- Environmental Protection Agency (U.S.)
F/Mv	- food to micro-organism ratio
Fe^{+++}	- ferric iron
g	- gram
h	- hour
ha	- hectare
kg	- kilogram
km	- kilometre
kW	- kilowatt
kWh	- kilowatt-hour
L	- litre

m	- metre
MCRT	- mean cell retention time
mg	- milligram
MJ	- megajoule
ml	- millilitre
MLSS	- mixed liquor suspended solids
mm	- millimetre
mm Hg	- millimetres of mercury
NH_4^+	- free ammonium nitrogen
O_2	- oxygen
OWR Act	- Ontario Water Resources Act
OWRC	- Ontario Water Resources Commission
P	- phosphorus
Q	- flow
s	- second
S.G.	- specific gravity
SRT	- solids retention time
SS	- suspended solids
SVI	- sludge volume index
SVR	- sludge volume ratio
SWD	- side wall depth
TKN	- total Kjeldahl nitrogen
W	- watt
WAS	- waste activated sludge

APPENDIX 'B'

GUIDELINES FOR COMPATIBILITY BETWEEN
SEWAGE TREATMENT FACILITIES AND
RESIDENTIAL LAND USES
(DRAFT POLICY NO. 07-05-01)

POLICY TITLE	GUIDELINES FOR COMPATIBILITY BETWEEN SEWAGE TREATMENT FACILITIES AND SENSITIVE LAND USES	NO. 07-05-01

Legislative Authority

the Ontario Water Resources Act, Sections 7(d) & 24
the Planning Act, Sections 14, 30(3), & 36

Statement of Principles

This policy is intended to minimize the effect of odours emanating from municipal and private sewage treatment plants and lagoons on sensitive adjacent land uses. The policy is an application of the Ministry's Land Use Compatibility Policy No. 07-03.

The separation distances specified in the policy are intended to mitigate the effects of offensive odours which may occur during normal daily operations or when facilities have minor overloads or upsets created by abnormal conditions or wastes. Since odour usually extends further than other environmental impacts associated with sewage treatment facilities, the separation distances will ensure adequate attenuation of any other environmental concerns.

1. Scope of Policy — The policy is applicable to waste stabilization ponds and sewage treatment plants. Plants are categorized into two classes: those with a design capacity equal to or less than 25,000 cubic metres of sewage per day (m^3/d) and those with a capacity greater than 25,000 m^3/d.

January 16, 1984

Point of Contact

Director, Environmental Approvals and Project Engineering Branch

Effective Date

September 7, 1983

The policy is not appropriate for dealing with the effects of major overloads or plant breakdowns on residential and other sensitive land uses.

2. Application

These guidelines apply to all Certificate of Approval applications under the Ontario Water Resources Act, Section 24, for new and expanding municipal and private sewage treatment facilities. The guidelines also apply to the advice that MOE provides to the Ministry of Municipal Affairs and Housing under the Planning Act. This relates to all development or redevelopment applications for residential or other odour-sensitive land uses adjacent to sewage treatment facilities.

3. Separation Distances

3.1 Residential Land Use

Where practical, residential uses should not be placed adjacent to treatment facilities.

3.2 Acquisition of Buffer Areas

When new facilities or major enlargements are proposed, an adequate buffer area should be acquired as part of a project in order to avoid imposing constraints on surrounding land use. Separation distances will be measured from the proposed odour-producing source to the facility lot line in this case.

Exceptions may be made when the future non-residential use of the adjacent lands is assured through such means as official plan designation and zoning, restrictive covenants in favour of the authority operating the plant or ownership by a co-operating public authority.

3.3 Alternatives to Buffer Area Aquisition

In the case where an adequate buffer area has not been purchased, the objective is to provide an optimum level of protection between sewage treatment facilities and residential structures. Reference may be made to the Land Use Plan Review Handbook, Chapter III-6 for guidelines on the measurement of separation distances.

When a buffer area cannot be provided for a sewage treatment plant, consideration should be given to covering sections of the plant and treating collected gases. A combination of distance, covering and treatment may, in some cases, be required.

3.4 Sewage Treatment Plants

3.4.1 Capacity Equal to or Less Than 25,000 m^3/d

The recommended separation distance is 150 metres. The minimum separation distance is 100 metres.

3.4.2 Capacity Greater Than 25,000 m^3/d

These plants will be dealt with on an individual basis and separation distance of greater than 150 metres may be required.

3.5 Waste Stabilization Ponds

The desirable separation distance varies from 100 to 400 metres depending on the type of pond and characteristics of the waste.

4. <u>Comments on Residential Applications</u>

In most cases, the Ministry of the Environment will concur with residential developments near sewage treatment facilties that have no history of objectionable odours, if the above guidelines are being met. If a facility does have a history of objectionable odours, a larger buffer zone may be required, at least until further abatement work has remedied the problem. Should any of the above conditions not be satisfied, the Ministry may advise against the proposed development.

Warnings may be applied to land titles or other legal documents relating to residential uses, which warn prospective buyers about the occasional nuisance effects of a nearby sewage treatment facility (see the <u>Land Use Plan Review Handbook</u> Chapter II-4, <u>Warnings Concerning Environmental Matters</u>).

APPENDIX 'C'

RECOMMENDED METRIC UNITS

APPENDIX 'C'

RECOMMENDED METRIC UNITS*

aeration mixing power requirements

- diffused aeration L/m.s (litres of air per cubic metre of aeration tank volume per second)
- mechanical aeration - W/m^3 (watts of aerator power per cubic metre of aeration tank volume)

air supply rate

- grit removal tanks - L/m.s (litres per metre of tank length per second)
- aerobic digesters - $L/m^3.s$ (litres of air per cubic metre of tankage volume per second)

area

- tankage, etc - m^2 (square metres)
- land - ha (hectares)

concentrations

- dilute - mg/L (milligrams per litre)
- concentrated - % (per cent)

conditioning chemical feed rate

- g/kg (grams of chemical per kilogram of sludge dry solids)

crop nitrogen requirements

- kg/ha.a (kilograms of nitrogen per hectare per year)

*- For a complete listing of metric units refer to WPCF Manual of Practice No. 6 [62]

depth-tanks, etc

- m (metres)
- rainfall - cm (centimetres)
- evapotranspiration - cm (centimetres)
- spray application - cm (centimetres)

detention time

- short - second or minute
- medium - hours
- long - days or years

energy

- MJ (megajoules)

food-to-microorganism loading rate (F/Mv)

- d^{-1} (per day, eg. kilograms of BOD_5 per day per kilogram of mixed liquor volatile suspended solids under aeration)

grit production

- mL/m^3 (millilitres of grit per cubic metres of sewage treated)

infiltration/inflow allowance

- mL/m.d)/m (millilitres per metre diameter per day per linear metre) of pipe length

organic loading rate

- aeration tanks - Kg $BOD_5/m^3.d$ (kilograms of 5-day biochemical oxygen demand per cubic metre of aeration tank per day)
- waste stabilization ponds - kg $BOD_5/ha.d$ (kilograms of 5-day biochemical oxygen demand per hectare of surface area per day)

oxygenation efficiency

- diffuser aeration - %
- mechanical aerators - kg O_2/kWh or kgO_2/MJ (kilograms of oxygen or kilograms of oxygen per megajoules)

per capita sewage flow rate

- L/cap.d (litres per capita per day)

permeability

- cm/s (centimetres of fluid flow per second)

pipe sizes

- mm (millimeteres)
- m (metres)

power

- W (watts)
- kW (kilowatts)

pressure

- positive or negative - kPa (kilopascals)

sludge production

- volume - L/m^3(litres of sludge per cubic metres of sewage treated
- mass - g/m^3 (grams per cubic metres of sewage treated)

solids loading (sedimentation, thickening and dewatering applications)

- kg/m^2.d (kilograms of solids per square metre of settling tank surface area, thickener area, or dewatering unit surface area per day)
- g/m.s (usually used for belt filter presses where the dimensional unit refers to the belt width)

spray application rate

- seasonal - L/ha.d (litres of effluent per hectare per day)
- instantaneous - cm/h (centimetres of effluent per hour)

surface settling rate (overflow rate)

- $L/m^2.s$ (lkitres of flow per square metre of tank surface per second)

tank dimensions

- m (metres)

temperature

- °C (degrees Celsius)

unit oxygen demand

- kg O_2/kg BOD_5 or N (kilograms of oxygen per kilogram of BOD_5 or nitrogen loading)

velocity

- fluid flow - m/s (metres per second)
- wind - km/h (kilometres per hour)

volatile solids loading (digestion applications)

- $g/m^3.d$ (grams of volatile solids per cubic metre of tankage volume per day)

volume

- small tanks - L (litres)
- large tanks - m^3 (cubic metres)

weir overflow rate

- L/m.s (litres of flow per metre of weir length per second)

APPENDIX 'D'

EXTRANEOUS FLOW ALLOWANCES

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EXTRANEOUS FLOW ALLOWANCES

In the design/assessment of any sanitary sewerage works facility there should be an allowance made for extraneous flows. (i.e., infiltration/inflow). The absolute value utilized in any specific system design/assessment will vary depending upon local conditions and/or the nature of the application of the infiltration allowance (i.e., sewer design; sewage pumping station design; sewage treatment plant design; existing sewerage works assessment and acceptance testing of new sewers).

In this Appendix the customary or design units are stated under each heading. Based on a typical plan of subdivision these customary units have been converted to "equivalents" for illustrative purposes only.

Acceptance Testing of New Sewers

Section MOE 02650, Clause 3.16 - Field Testing of the Ministry's Standard Specification for the Construction of Sewer and Watermains lists an allowable extraneous flow/leakage (infiltration/exfiltration) of 0.075 litres/millimetre diameter per 100 metres of sewer per hour.

This "customary" unit converts to the following based on the typical plan of subdivision.

a) 22 L/cap.d
b) 0.01 L/ha.s

Sewer Design

Typically, in the design of a sanitary collection sewer system a peak extraneous flow allowance of between 0.10 and 0.28 L/ha.s is made. These customary units, when applied to a typical plan of subdivision convert to the following values.

a) 0.72 to 2.03 L/mm Ø/100 m/h*
b) 212 to 593 L/cap. d

* total sewer system including main sewers, service connections and building sewers.

The above-noted design value is for new collector sewer systems and assumes;

a) Strict control by the municipality of building sewer connections (i.e., no roof drains or foundation drains connected directly or indirectly to the sanitary sewers).

b) Adequate design and inspection during the construction of the public sewers and the private connections.

c) A routine inspection and maintenance programme will be undertaken by the municipality/operating authority to ensure that a "tight" sewer system is maintained.

Sewage Pumping Stations and Sewage Treatment Works

As with the design of new sanitary collector sewers it is accepted practice to make an allowance for extraneous flows in the design of any sewage pumping station or

sewage treatment facility. However, as the design period for pumping stations and treatment facilities is generally less than that of the sewers (i.e., 10-20 years vs 20-40 years) a lesser extraneous flow allowance should be used. Also, while the allowance is made in sewer design it is assumed that the actual volumes received will be substantially less because of the controls and inspections which are undertaken during and after construction.

Therefore, in the design of any new pumping station or treatment facilities complementary to a new collector sewer system an extraneous flow allowance of 90 L/cap. d (average) and 227 L/cap.d (peak) should be made.

This design value, when applied against the typical plan of subdivision, is approximately equivalent to;

a) 0.043 - 0.107 L/ha. s
b) 0.308 - 0.776 L/mmØ/100 m/h

Assessment of Existing Sewage Works

The capital and operating costs associated with new sewerage works facilities are increasing steadily. In addition, the Ministry's "Water Management - Goals Policies, Objectives and Implementation Procedures of the Ministry of the Environment" requires that all Certificates of Approval for new sewage treatment facilities contain the effluent requirements for the facility.

Accordingly, studies to ascertain the extent and source of extraneous flows are becoming more important.

Experience in the United States has indicated that if the extraneous flow, based upon the highest weekly average

within a 12 month period, is less than 140 L/mm.km.d, rehabilitation of the sewer system will not be economical.

Based upon the preceding typical plan of subdivision this value of 140 L/mm.km.d is approximately equivalent to

a) 0.08 L/ha.s
b) 171 L/cap.d

NOTES:

1. The "typical" plan of subdivision has the following characteristics.

Overall Area -	23 ha
Total Lots/Units -	263
Typical Set Back -	7.6 m
Population Density -	3.0 persons/lot (unit)
Main Sewer length and size -	3072 m of NPS-8*
Sewer lateral length and size -	2645 m of NPS-5*
Building sewer length and size -	2004 m of NPS-4*

*Nominal pipe size is indicated with a NPS designator number.

2. Critical in reducing the volume and rate of flow to be handled by a foundation drainage system and hence, its ability to keep a basement dry is lot grading. Therefore, in all new development, every effort should be made to ensure that the lot is drained away from the foundation walls.

APPENDIX 'E'

CHLORINATION REQUIREMENTS

STORAGE AND USE OF CHLORINE

1. INTRODUCTION

1.1 Chlorine has wide use in industry. 68kg (150 lb) cylinders are commonly used. To minimize handling where usage is large, containers may be linked by a manifold system; 1Mg (1 ton) containers, or railroad tank cars from 16 to 90Mg (16 to 90 tons) capacity may also be used.

2. PROPERTIES

2.1 Chlorine liquid has a relative density of 1.47 (water = 1).

2.2 Chlorine gas is greenish-yellow in colour and is about 2½ times as heavy as air.

2.3 A volume of liquid chlorine when vaporized will yield about 460 volumes of gaseous chlorine at $0^{\circ}C$ ($32^{\circ}F$).

2.4. At 0°C (32°F) and 1 atmosphere pressure, 1 kg (2.2 lb) of liquid chlorine will vaporize into 311 L (10.98 ft^3) of gaseous chlorine.

2.5 Dry chlorine does not corrode steel or common metals at ordinary temperatures. In the presence of moisture, hydrochloric acid is formed causing corrosion.

2.6 Carbon monoxide and chlorine react in sunlight to produce an extremely poisonous gas - phosgene.

2.7 Ammonia with excess chlorine may form the violently explosive - nitrogen trichloride.

2.8 Chlorine is non-flammable, but like oxygen, supports combustion. Some of these reactions are explosive in intensity, e.g., with hydrogen, acetylene, ammonia, turpentine, most hydrocarbons, alcohols, ethers, and finely divided metals.

3. HEALTH HAZARDS AND TRAINING

3.1 Chlorine has an exceedingly disagreeable odour and is a powerful respiratory irritant. Odour is detected at 3.5 ppm and irritation is noticeable at 15 ppm. Long exposure at the odour threshold may produce serious physiological disturbances. Exposures of 30 to 60 minutes at concentrations of 40 ppm are dangerous. A few inhalations at concentrations of 1,000 ppm are fatal. (ppm = parts per million)

3.2 The American Conference of Governmental Industrial Hygienists has established the following chlorine concentration levels in the working enviroment for 1980:

(i) Time Weighted Average (TWA) for an 8-hour workday or 40-hour work week - 1 ppm.

(ii) Short Term Exposure Limit (STEL) for a 15-minute continuous exposure - 3 ppm.

3.3 Persons handling chlorine must be competent and trained in the handling of the equipment and the use of personal protective equipment and other necessary safeguards.

4. STORAGE AND USE OF CHLORINE

4.1 The location for storage or use of chlorine should preferably be in an isolated building or in a room without direct access to the rest of the building. Construction must be of fire resistant or non-combustible material and the building or room shall have a concrete floor with good drainage.

4.2 Chlorine shall not be stored below ground level.

4.3 Chlorine shall not be stored with combustible materials.

4.4 The container shall be protected against excessive external heat sources, dampness and mechanical damage. Outside storage should be sheltered from the direct rays of the sun.

4.5 Separate storage spaces shall be provided for full and empty containers.

4.6. The areas for storage or dispensing of chlorine shall be located where escaping gas cannot accidentally enter the building's ventilation system. Cylinders under 68kg (150 lb) will be stored in an upright position and securely fastened. The valve outlet caps and protection hoods shall be in place on all cylinders not in use.

4.7 When a scale room is used, it should be adjacent to the storage area and be of non-combustible or fire resistant construction.

4.8 Chlorine storage areas and scale rooms shall be clearly marked "DANGER! - CHLORINE!".

5. STORAGE OF 1Mg (1 ton) CONTAINERS

5.1 Paragraphs 4.1 to 4.8 shall apply.

5.2 1Mg (1 ton) containers shall be stored horizontally on fixed rollers or well chocked on "I" beams with the two outlet valves in a vertical plane, as recommended by the Chlorine Institute Incorporated.

6. RAIL CAR STORAGE

6.1 A special licence by the Board of Transport Commissioners for Canada is required for transfer of chlorine from rail cars to plant storage tanks. The plant storage tank installation shall be reviewed by the Industrial Safety Branch, Ministry of Labour.

6.2 A dead end siding restricted to chlorine tank cars shall be provided.

6.3 Tracks shall be level and the siding shall be protected by a locked derail or a locked closed switch.

6.4 The area shall be well and clearly posted "DANGER! - CHLORINE!".

6.5 Chlorine sidings shall be located at least:

(i) 600m (2000 ft.) from public buildings and areas of public congestion.

(ii) 300m (1000 ft.) from private residences.

(iii) 23m (75 ft.) from storage or transfer facility for a combustible or dangerous commodity.

(iv) 15m (50 ft.) from an adjoining property or from the curb line of a main roadway or street.

7. EXITS

7.1 The exit doors shall be hinged to open outwardly. There shall be two or more means of egress if the distance of travel to a single means of egress exceeds 4.5m (15 ft) (O.Reg. 844/79, Sec. 123).

7.2 The distance of travel to the nearest of two or more means of egress shall not exceed 23m (75 ft) (O.Reg. 844/79, Sec. 123).

7.3 Exits shall lead directly outdoors. Interior access to a non-chlorine area shall only be through a pressure ventilated vestibule.

8. VENTILATION

8.1 Continuous mechanical ventilation at the rate of 3 air changes per hour shall be provided or screened openings to the outdoors shall be provided within 15cm (6 in) of the floor in the ratio of 0.2 percent of floor area. Similar openings shall be provided near the ceiling. The openings shall be distributed to produce the maximum air circulation across the floor.

8.2 Provision for emergency mechanical ventilation should be made sufficient to produce 30 air changes an hour taking suction at a maximum of 1m (3 ft) above floor level and discharging at least 2.5m (8 ft) above grade and away from air intakes.

8.3 A switch for the emergency mechanical ventilation shall be located outside the chlorine room entrance door or in the room adjacent to the chlorine room.

9. MATERIALS OF CONSTRUCTION

9.1 All materials of construction, details of installation and operating procedures shall be as recommended by the chlorine supplier and shall be reviewed by the Ministry of Labour prior to installation.

9.2 Vaporizers, evaporators and pressure piping shall fulfil the Boilers and Pressure Vessels Act.

10. PROTECTIVE EQUIPMENT

10.1 Full face, self-contained or air-supplied respiratory protective equipment, suitable for use in a chlorine atmosphere for a period of not less than 15 minutes, should be supplied and be readily available.

10.2 Full face emergency canister gas masks of a type approved for chlorine service shall be readily available where chlorine is stored or used.

10.3 The respiratory protective equipment shall be kept in dust-tight cabinets in a conspicuous location outside the area of probable contamination. The equipment must be inspected regularly and over-age canisters replaced as recommended by the supplier.

10.4 Only self-contained or air-supplied types of respiratory protective equipment shall be used where the chlorine concentration may be above 1 percent (10,000 ppm). Gas masks are of no use in these cases.

10.5 Protective goggles, aprons, gloves and safety footwear shall be available for persons loading, storing or handling chlorine.

10.6 Eye wash fountains designed for easy operation shall be located within a 10 second travel distance and outside the area of probable contamination. The distance between the chlorine room exit door and the eye wash fountain should be less than 4.5m (15 ft).

11. CHLORINE LEAKS

11.1 A plan of action shall be prepared to deal with emergency leaks. A chlorine tool kit, as recommended by the supplier, shall be available.

12. FIRST AID

12.1 Remove the affected person from the contaminated area. Keep him warm and quiet. Lay the victim on his stomach with his head and chest slightly lowered. If the victim is conscious, do everything possible to discourage coughing. Oxygen is of great value. Even in mild cases, inhalation of oxygen relieves chest irritation. In severe exposure cases, oxygen should be administered until the victim is able to breathe easily. Contaminated clothing should be removed and the contaminated body areas flushed with water. If breathing seems to have stopped or has ceased, apply artificial respiration without delay along with oxygen. The services of a physician should be obtained as quickly as possible.

13. CHLORINE DISPOSAL

13.1 Chlorine may be absorbed in solutions of caustic soda, soda ash or hydrated lime. A solution of caustic soda absorbs chlorine most readily.

RECOMMENDED SOLUTIONS FOR ABSORBING CHLORINE FOR A 1Mg (1 ton) CONTAINER

ABSORBING CHEMICAL	WATER
Caustic Soda -- 1200kg (2500 lb)	3600L (800 gal)
Soda Ash -- 2700kg (6000 lb)	9000L (2000 gal)
Hydrated Lime -- 1200kg (2500 lb)	11350L (2500 gal)

REFERENCES: Chlorine Manual, The Chlorine Institute Incorporated.

CHLORINATION OF POTABLE WATER SUPPLIES

Ministry of the Environment
Ontario

BULLETIN 65-W-4

POLLUTION CONTROL BRANCH

Revised August, 1980.

CHLORINATION OF POTABLE WATER SUPPLIES

INDEX

ONTARIO MINISTRY OF THE ENVIRONMENT

BULLETIN

CHLORINATION OF POTABLE WATER SUPPLIES

1.0 INTRODUCTION

1.1 Purpose of Bulletin

Disinfection, to kill pathogenic organisms, is the most important step in any water treatment process. In Ontario it is usually accomplished by adding chlorine. This chemical has many other uses in water treatment such as coagulation aid, taste and odour control and maintenance of water quality in the distribution system, but its primary purpose is disinfection. This bulletin outlines the requirements to achieve adequate disinfection and the procedures to follow when it is not achieved. The bulletin also outlines a design standard. New installations should meet the criteria as set out in the bulletin and existing facilities should be brought up to these standards.

1.2 When is Disinfection Required?

Continuous and adequate disinfection is required when the supply is obtained from a surface source; when ground water sources are or may become contaminated, as in fractured limestone areas; when the supply is exposed to contamination during treatment or when emergency conditions such as flooding or epidemic, indicate the need. Disinfection equipment should also be available at those plants where continuous disinfection is not required, to allow temporary disinfection if unsatisfactory or poor bacteriological quality of water is reported. The design of all plants should incorporate suitable connections for disinfecting equipment to be added for this purpose.

1.3 Types of Chlorination

Chlorine when added to water immediately dissociates into hypochlorous acid and hydrochloric acid. The former compound can

further dissociate to hypochlorite ion and hydrogen. Hypochlorous acid predominates when the pH is below 7.5. It is the compound that is the prime disinfecting agent in a free chlorine residual. However it is very reactive and will quickly combine with certain compounds (eg. ammonia) and slowly with many other compounds, that may be present in water, to produce a combined chlorine residual (monochloramine, dichloramine etc.). When sufficient chlorine is present so that the reactions that form the combined chlorine are completed, the breakpoint has been reached. The addition of more chlorine will then yield a free chlorine residual.

Of the many regimes of chlorination, simple or marginal chlorination is probably the most common. This also probably the least effective, especially if it is the only treatment applied to surface water. Chlorine is applied to give an initial total chlorine residual of 0.2 - 0.5 mg/L, predominantly as a combined residual that frequently disappears in the distribution system.

Free residual chlorination produces a much superior disinfecting agent. Sufficient chlorine should be added so that the free residual comprises about 60 to 80 percent of the total residual and it should be maintained through all of the water treatment plant and distribution system. Very high free residuals (super-chlorination) would necessitate at least partial dechlorination, before entering the distribution system, with sulphur dioxide, sodium thiosulphate or activated carbon. The addition of ammonia will produce the more stable but less active disinfectant, chloramine.

If only a chloramine residual is desired, it can be achieved by adding ammonia before the chlorine. However a much higher dose and/or much longer contact time is required to achieve the same degree of disinfection as a free chlorine residual.

Pre-chlorination together with post-chlorination, as required is a frequent mode of operation at Ontario water treatment plants. If organic compounds in the raw water tend to cause formation of chlorinated organics (chloroform, etc) it may be advisable and possible to chlorinate just prior to or after filtration when many organic

precursors may have been removed. However, the bacteriological integrity of the water must receive first priority when considering any modification to chlorination practices aimed at reducing the formation of chloro-organics.

2.0 EQUIPMENT

Chlorination equipment must be readily available at all water treatment plants. This includes all ground water supplies where chlorination is not continuous.

2.1 Capacity

Chlorination equipment shall have a maximum feed capacity at least 50% greater than the highest expected dosage required to provide a free chlorine residual of 1.0 mg/L in the finished water.

2.2 Duplicate Equipment

Chlorine feed equipment (both gas and hypochlorite chlorinators) at waterworks where disinfection is required, shall be installed in duplicate, to provide uninterrupted chlorination in the event of a breakdown. In addition, spare parts consisting of at least the commonly expendable parts such as glassware, rubber fittings, hose clamps and gaskets, should be provided for effecting emergency repairs.

For a multi-well supply system requiring chlorination for disinfection to standby requirements may be met by one portable unit.

2.3 Chlorinators and Controls

Dependable feed equipment, either of the gas feed or positive displacement solution feed type, shall be used for adding chlorine. Automatic proportioning of the chlorine dosage to the rate of flow of water should be provided at all plants, especially where the rate of flow varies without manual adjustment, or operation of valves and/or switches. Where the chlorine demand is not constant, it may also be necessary to either adjust the chlorine dose through

a chlorine residual analyzer, or feed a constant chlorine dose and use the chlorine residual analyser to regulate the feed of a dechlorinating agent.

2.4 Gas Chlorination

2.4.1 Building Design

Gas chlorine equipment - (chlorinators, weight scales, chlorine cylinders) must be located in an isolated room or rooms. In larger installations the storage and weighing facilities should be in a room separate from the chlorinators. The construction of the facility should be of fire-resistant and corrosion proof material, have concrete floors and be gas tight. All interior surfaces should be coated with a substance impermeable to chlorine gas.

A set of corrosion resistant scales should be available for weighing the chlorine cylinders. Scales for 69 kg (150 lb.) cylinders should be of the low profile type. Non-low profile scales shall be recessed in the floor. Safety chains shall be used to retain each cylinder, in storage and on weigh scales, in a safe upright position.

Chlorine should not be stored below ground level and the cylinders must be protected from excessive heat, dampness and mechanical damage. One ton cylinders shall be stored on their sides on level racks.

Where rail cars are used, a dead end siding restricted to chlorine tank cars shall be provided. The tracks must be level and protected by a locked derail or a locked closed switch.

Areas containing chlorine or chlorination equipment shall be clearly marked, "DANGER! CHLORINE STORAGE" or, "DANGER! CHLORINE FEED EQUIPMENT" as applicable. The exit doors with panic hardware shall be hinged to open outwardly. There shall be two or more exits if the distance to travel to the nearest exit exceeds 15 feet. All exits from the chlorine room and storage area should be to an outside wall. Access between these rooms is permitted if they have a common wall.

The temperature in the chlorine storage and scale room shall not be higher, and preferably slightly lower than that in the chlorinator room. The gas lines between the scales, chlorinators and injectors shall not be located directly on an outside wall or in a location where low temperatures may be encountered.

2.4.2 Safety Equipment

Each plant shall have readily available, a self-contained or air-supplied respirator of the pressure demand type. One respirator shall be located in a conspicuous location outside the area of probable contamination.

Protective clothing including gloves, goggles and safety shoes shall be available for persons handling chlorine.

Eye wash fountains shall be located as near as possible but outside the area of probable contamination.

All chlorine rooms must have chlorine leak detector alarm system.

Container emergency kits to repair leaking valves, fusible plugs or the tanks themselves are available from chlorine suppliers. There are different kits for each size of tank and the proper size should be available at each water plant.

2.5 Hypochlorite Chlorination

It is important that hypochlorite compounds which contain an algicide not be used as a disinfecting agent in potable water systems.

2.5.1 Safety Procedures

Sodium hypochlorite, (a liquid) and calcium hypochlorite (a powder) are frequently used to provide chlorination at small municipal water plants and to disinfect mains and reservoirs. Certain safety precautions must be observed in the storage and handling of these compounds.

Calcium Hypochlorite

Certain precautions must be taken when adding granular calcium hypochlorite.

1) Store containers in a clean, cool, dry area away from any combustible material. Spontaneous combustion can result from improper storage. Keep the containers away from moisture, heat and fire. There should be no smoking in this area.

2) Metal drums should be kept upright and should not be dropped, rolled or skidded. Calcium hypochlorite, if dropped, can explode and burn.

3) Empty containers should be thoroughly rinsed with water.

4) When handling calcium hypochlorite it must never have contact with the eyes and it can cause serious burns in the lungs or on damp skin. Face shields with dust masks together with long gloves and other protective clothing must be worn.

5) When measuring calcium hypochlorite a plastic, glass or enameled device that is clean and dry must be used. It should only be mixed with water.

Sodium Hypochlorite

Sodium hypochlorite is much safer than calcium hypochlorite but does require much more storage space and is more costly to transport long distances.

1) There is no fire hazard from the storage of sodium hypochlorite but corrosion from spillage can be a problem if the facilities are not corrosion resistant and cannot be well flushed with water.

2) When handling the chemical, proper clothing (gloves, eye goggles, etc.) shall be worn.

2.5.2 Chlorination Procedures

Where a powdered product is used, hypochlorite solution shall be prepared in a separate tank to allow clarification by settling before it is directed to the solution storage tank serving the hypochlorinator. If the water used to dissolve the granular hyochlorite has a hardness in excess of 100 mg/L, the water should be softened with hexametaphosphate (Calgon) or an ion exchange unit. Periodic purging of the metering system, with muriatic acid, may be necessary to remove calcium deposits. The acid must be flushed from the system before it is put back into use.

The stability of the hypochlorite solution is increased if the concentration is low; the pH is above 10; iron, copper and nickel content is low; and the solution is stored in the dark at low temperature.

2.6 Chlorine Residual Testing

It is important that all surface water supplies be equipped with a continuous chlorine residual analyzer and recorder as well as a continuous turbidity analyzer and recorder; this is especially so at larger plants and where water near the intake could become polluted. All surface water plants should at least be equipped with an alarm system that would indicate when the chlorination equipment malfunctions.

Ground water sources, where poor water quality and/or minimum supervision indicates a possible health hazard, should have an automatic chlorine residual analyzer and recorder equipped with a high and low residual alarm or at least an alarm system that would indicate when the chlorination equipment malfunctions.

All installations must be equipped with a permanent standard chlorine residual testing device. It is preferable to use a DPD comparator test kit, an amperometric titrator or equivalent. The amperometric titrator can be used to check the accuracy of a

continuous chlorine residual analyzer. The above methods can be used to measure a free chlorine residual in the finished water, the distribution system or in the stand pipe when an emergency or other circumstances require a free residual.

3.0 ROUTINE OPERATION

3.1 Chlorine Residual

3.1.1 General

Chlorine can be present in water as either a free or a combined residual. The bactericidal effectiveness of both residual forms is markedly reduced by high pH or turbidity, while it is enhanced by a higher temperature or a longer contact time. A free chlorine residual, while a much more effective disinfectant, also readily reacts with ferrous iron, manganese, sulphides and organic material to produce compounds of no value for disinfection.

3.1.2 Requirements

For surface water treatment plants achieving low uniform turbidities (1 FTU or less) with a minimum of 2 hours of chlorine contact or for ground water supplies proven free of hazardous bacterial or viral contamination but still requiring chlorination, the minimum total chlorine residual shall be 0.2 mg/L. For all other chlorinated supplies the minimum total chlorine residual shall be 0.5 mg/L. These are minimum acceptable residuals not target or objective residuals. A minimum contact time of 15 minutes (preferably 30 minutes) before the first possible consumer shall be provided at all times. The chlorine residual shall be differentiated into its free and combined portions. It is preferable that most of the residual be a free residual. Adequate disinfection may not occur at these minimum levels if the pH is above 7.5 or the turbidity above 1 FTU.

As circumstances demand, the minimum requirements for chlorine residual and/or contact time may be increased.

The chlorine residual test must be performed as frequently as needed to ensure that an adequate chlorine residual is maintained at all times. Such considerations as raw water quality and the resultant variation in chlorine demand, and changing flow rates must be taken into account.

The accuracy of an automatic chlorine residual analyzer shall be checked daily. This shall be accomplished using the amperometric titrator. The results of the check shall be inscribed on the recording chart along with the date and operator's initials opposite a mark indicating the time of the check.

A chlorine residual should be maintained in all parts of the distribution system. This will do little to protect the supply in the event of a main break or some other disaster but should control nuisance growths. The residual should be differentiated into its free and combined portions. The pH of the sample should also be recorded so that the major chlorine constituents in the water can be determined.

The amount and type of chlorine residual present when routine bacteriological samples are taken should be recorded, because this allows a more complete evaluation of the condition of the distribution system.

3.1.3 Determination

A representative sample of chlorinated water should be tested. From a tap, the water should be kept running for 5 minutes before taking the sample.

The time when the chlorine residual test should be made depends on where the sample was taken. If the sample has just been chlorinated it should be held for 15 minutes to simulate the minimum contact time, in a covered demand-free container away from light and heat. However, a sample from the distribution system or finished water after a contact chamber should be tested immeiately.

Determination of a chlorine residual should be done by one of the methods outlined in the most recent Standard Methods (14th Edition, 1976) which are preferable to the regular orthotolidine test which has been used extensively. At present the most widely accepted methods are the DPD (diethyl-p-phenylene diamine), both titrimetric and colorimetric, and the amperometric titration. For small water treatment plants or field testing a DPD comparator kit is acceptable.

When using the DPD colorimetric (comparator) test a few important procedures must be observed.

1) The glass cell must be thoroughly rinsed after each test, since any trace of the potassium iodide (Tablet #3) will cause the chloramine colour to develop in the next test for free chlorine.

2) To facilitate dissolving the tablets, they can be crushed while still in their tinfoil packets.

3) Disintegrate tablet #1 in a few drops of the sample in the test cell. Fill the test cell to 10 ml and mix rapidly.

4) The free chlorine residual must be read within 30 seconds of adding the sample to the cell.

5) The total chlorine residual is determined by adding tablet #3 (crushed) to the same sample in the test cell, mixing, waiting 2 minutes for the full colour to develop and then reading the results in the comparator.

3.2 Records

Minimum records shall include:

1) Daily records of the chlorine used and scale readings.

2) Results from all chlorine residual test, together with the flow rate and chlorine feed rate and the time of testing.

3) The daily water consumption and the chlorine dosage in milligrams per litre.

4) Details on chlorine cylinder changes, orders and chlorine on hand.

5) Monthly and yearly summaries of chlorine consumption and feed rates.

6) For surface supplies, daily air and water temperatures and weather conditions eg. rain, cloud, sunny, snow etc. together with wind direction and strength.

4.0 EMERGENCY OPERATION

At all facilities supplying municipal drinking water, a procedure to follow in case of emergency (ie. plant malfunction) must be developed. A list of procedures for the operator to follow must be posted in a prominent location in the plant.

This list must include.

1) The order not to pump unchlorinated or inadequately chlorinated water to the distribution system.

2) The name, address and telephone number of:

   a) Senior supervisory personnel,

   b) Medical Officer of Health and an alternative in the regional health unit if the Medical Officer of Health cannot be reached,

   c) The local M.O.E. District Officer and an alternate,

   d) Chlorinator service company (to be called only if chlorinator needs servicing),

   e) Chlorine supplier (to be called when chlorine required or when tanks malfunction).

3) The exact procedure to follow in order to increase the total chlorine residual leaving the plant to a minimum of 1.5 mg/L.

Wherever chlorination is required, the Ministry of Environment and the Medical Officer of Health must be notified immediately if unchlorinated or inadequately chlorinated water (total residual below 0.2 or 0.5 mg/L or level required) is directed to the distribution system. If this has occurred the Ministry of Environment may require the chlorine feed rate to be increased to provide a 1.0 mg/L or higher residual leaving the plant. Extensive flushing may also be required to carry the residual through the distribution system. Depending on the circumstances additional steps may be required.

When the chlorine residual is increased all customers who may be adversely affected must be notified.

5.0 ADVERSE BACTERIOLOGICAL RESULTS

When the results from bacteriological samples collected from the distribution system indicate unsatisfactory water quality on the basis of the Ontario Ministry of Environment Drinking Water Objectives, (presence of fecal coliform bacteria or the numbers of coliform bacteria five or more per 100 ml) the procedures to follow immediately are:

1) Notify the Ministry of the Environment (increased chlorine residuals may be advised),

2) Collect further samples to confirm the results and determine the extent of the contamination. Chlorine residuals should also be recorded.

If these samples still show unsatisfactory water quality, the Medical Officer of Health and the Ministry of the Environment must be notified and the chlorination increased to provide a total

chlorine residual of 1.0 mg/L or a free chlorine residual of 0.2 mg/L at the end of the distribution system. Systematic flushing or swabbing may be necessary in order to achieve and maintain a residual at the ends of the distribution system.

A thorough study of the treatment plant and/or distribution system should be undertaken to determine the cause of the adverse bacteriological results. If the conditions warrant it the Ministry of Environment should recommend to the Medical Officer of Health that a boil-water advisory be issued.

When the bacteriological samples indicate poor water quality (coliform bacteria present at levels below five per 100 ml in more than 10% of the monthly samples or other indicator bacteria (see MOE Drinking Water Objectives) the Ministry of the Environment may recommend some of the following procedures:

1) Initiate chlorination on an unchlorinated supply,

2) Increase the chlorine residual requirements in the finished water of 1.5 mg/L or more, and maintain the level until notified by MOE,

3) Establish a total or free chlorine residual to the end of the distribution system,

4) Disinfect the distribution system as for new mains (Sec. 6.2),

5) Undertake a thorough resampling of the distribution system which should continue until the water quality is again acceptable.

6.0 DISINFECTION OF NEW AND REPAIRED MAINS

6.1 Preparation

Chlorine is predominantly a surface active disinfectant that will not penetrate debris rapidly to kill microorganisms. This

debris may also react with the chlorine to reduce its disinfecting power. For these reasons, prior to disinfection of new or repaired works, all the debris must be removed. This can be achieved by extensive flushing with potable water, preferably with foam swabs.

6.2 Disinfection of New Water Mains

There are three procedures outlined in the AWWA Standard C601-68.

a) Continuous Feed Method

After the main has been cleaned, potable water with a chlorine residual of at least 50 mg/L is fed into the main until it is full. This is achieved by having a constant flow rate and injecting a hypochlorite solution into the main with a hypochlorinator or using liquid chlorine through a solution-feed chlorinator and booster pump. The chlorinated water should remain in the pipe for a minimum of 24 hours, during which time all valves and hydrants are operated to ensure their disinfection. At the end of the 24 hour period, the chlorine residual must be no less than 25 mg/L or the procedure must be repeated.

b) Slug Method

This method is suitable for large, long mains where continuous feed is impractical. Following cleaning, potable water is fed into the main at a constant rate. Chlorine is added to the water at a constant rate so that the resulting residual is no less than 300 mg/L. The chlorine dosage is continuous for a sufficient period to ensure that the minimum contact time is 3 hours. As the chlorinated water flows past, all valves and hydrants etc. must be operated to ensure their disinfection.

c) Tablet Method

This method is best suited to short, small diameter mains (up to 30 cm (12 inches)). Since the preliminary cleaning must

be forgone it is absolutely essential that during construction the pipe interior remains clean and dry. The calcium hypchlorite tablets must be placed at the top of the pipe using an approved adhesive. The main is slowly filled (flow less than 0.3 m/s 1 ft/s) to prevent washing the tablets to the end of the main. Sufficient tablets must be used to result in a final chlorine residual in excess of 50 mg/L. The contact time is a minimum of 24 hours after which the residual must be about 25 mg/L. If the water temperature is below $5^{\circ}C$ the contact time must be increased.

6.3 Disinfecting Repaired Water Mains

When a leak is minor and the water in the main always has a positive pressure, no disinfection is required after the repair is complete. However, with a more serious break the main must be disinfected before being put back into service. AWWA Standard, C601-68 lists two alternatives. If the broken main need not be put back into service immediately the methods outlined for new mains would ensure better disinfection.

a) Swabbing and Flushing

This procedure is the minimum that may be used. The interior of all pipes and fittings must be swabbed with a 5% hypochlorite solution as they are installed. The chlorine solution can be sprayed on with a small pressurized tank. This is followed by flushing, preferably in both directions, until the coloured water is eliminated.

b) Slug Method

Where possible, the following method should be used. The main with the break is isolated and repaired, then flushed and if necessary foam swabbed to remove all debris. Chorine is then introduced, as in Sec. 6.2 b), except that the residual may be increased to 500 mg/L and the contact time reduced to ½ an hour.

After the contact period the main is well flushed and then put back into service.

6.4 Bacteriological Testing

After the new or repaired main has been well flushed with potable water, to remove the heavily chlorinated water, bacteriological samples must be taken to test the effectiveness of the disinfection. If the main is very long several samples should be collected along its length. In distribution systems that normally carry a free chlorine residual one sample or a set of samples (in a long main) is sufficient. However in all other distribution systems a second sample or set of samples should be collected after 24 hours. A main is considered adequately disinfected if there are no detectable coliforms in any of the samples.

New mains must not be put into service until the coliform tests are acceptable. The disinfection process must be repeated if they are not.

If possible, repaired mains should also be kept out of service until acceptable results are received. This is seldom possible but if fecal contamination of the main is known or suspected it must be done to prevent a public health hazard. If the test sample(s) are positive for coliforms the disinfection should be repeated.

7.0 DISINFECTION OF NEW OR REPAIRED RESERVOIRS

7.1 Preparation

As with water mains, the interior of storage facilities must be cleaned and free of debris before attempting the disinfection process. This is accomplished by washing down the walls and floors with high pressure jet cleaning equipment and/or long handled brushes. All the debris must be rinsed from the tank interior before disinfection.

7.2 Disinfection Procedures

Three methods for disinfection are as follows:

a) First Method

This is suitable for tanks where gross contamination has occurred. The tank is filled with potable water to which has beed added, early in the process, sufficient chlorine to result in a 50 mg/L residual when the tank is full. The tank is left for at least 6 hours, preferably 24 hours, then drained to waste and refilled from the regular supply.

b) Second Method

This method can be used in a relatively clean reservoir, such as following routine cleaning or repair. The walls, floor and stanchions are sprayed with a 200 mg/L chlorine solution. The tank is well flushed, filled with potable water from the distribution system and put into service.

c) Third Method

Water containing 50 mg/L chlorine is placed in the tank to such a depth that when the tank is filled the resultant chlorine concentration is no less than 2 mg/L. The water containing the 50 mg/L chlorine is held in the tank for 24 hours before the tank is filled. The full tank in turn is allowed to stand for 24 hours after which the tank may be put into service without draining the water used to disinfect it.

7.3 Bacteriological Testing

After the new or repaired reservoir has been filled with potable water, bacteriological samples must be taken to ensure adequate disinfection. If a free chlorine residual is usually carried in the system only one set of samples is required but in all other systems a second set of samples should be collected after 24 hours.

The bacteriological samples must show no detectable coliforms before a reservoir is put into service. If coliform bacteria are detected the disinfection process must be repeated until satisfactory results are obtained.

8.0 DISCHARGE OF CHLORINATED WATER

Chlorinated water, as used in the disinfection of water mains and reservoirs, can be very toxic to aquatic organisms and it should not be disposed of without careful thought to its effect on the receiving water or sewage treatment plant (STP).

Chlorinated water can be discharged to:

1) Sanitary Sewers - This is a safe course to follow especially if the volume is not great and there is a considerable distance from the point of addition to the STP. However if there is a large volume, eg. with a reservoir, it is essential to contact the municipality to ensure that the operation of the STP is not adversely affected by a hydraulic overload or a massive slug of water with a high chlorine residual.

2) Receiving Waters - This can be detrimental to aquatic life and many fish kills have resulted. Water with a free chlorine residual should not be discharged to a stream or lake. If a combined chlorine residual is present, the concentration at the edge of the mixing zone (where allowed) should be below 0.002 mg/L.

3) Storm Sewer - This should be thought of as directly connected with the receiving water and the same restrictions should apply, even though there could be considerable dilution during wet weather.

4) Drainage Ditch - Discharge to an open ditch is a good alternative, especially if the point of addition is a

considerable distance from the receiving water and the ditch is unlined and is full of weeds and other organic material. Sunlight and high temperatures would help to dissipate the chlorine quickly.

If the above conditions cannot be met, a slow discharge of the chlorinated water to a sanitary sewer or ditch can be used. This is easier and cheaper than dechlorination. If dechlorination is necessary (ie. with direct discharge to a small stream), there are several chemicals that can be used effectively. Adequate mixing and dosage of the chemical with the chlorinated water must be ensured. The amount of dechlorination chemical required can easily be determined from the following equation.

Excess chlorine residual x Factor = Dechlorination chemical required

This can be worked out in mg/L, lbs or whatever units are appropriate.

There are five chemicals that can be used to dechlorinate the water:

1) Hydrogen Peroxide - (Factor = 0.479) - This is probably the best chemical when discharging to an environmentally sensitive watercourse. It is cheap and an overdose will only add more oxygen to the stream.

2) Sulphur Dioxide - (Factor = 0.901) - This chemical is cheap but it will slighly lower the pH in the receiving water.

3) Sodium Thiosulphate - (Factor = 2.225) - This will cause some sulphur turbidity but an excess is harmless.

4) Sodium Sulphite - (Factor = 1.775) - Excess will lower the dissolved oxygen in the stream.

5) Sodium Pyrosulphite - (Sodium Metabisulphite) - (Factor = 1.338) - Excess will lower the dissolved oxygen in the stream.

For example, a total chlorine residual of 21 mg/L measured in a disinfected water main of 11,000 L (2400 gal) for discharge at only 1 mg/L Cl_2 could be neutralized with hydrogen peroxide. The dosage required would be 20 mg/L x 0.479 = 9.6 mg/L and the total amount needed would be 9.6 x 11000 = 105.6 gm of H_2O_2. This would represent, in terms of 35% commercial grade hydrogen peroxide (sp. gr. 1.13 g/ml), $105.6 \times \frac{100}{35} \times \frac{1}{1.13}$ = 267 ml of concentrate.

APPENDIX 'F'

REFERENCES

APPENDIX 'F'

REFERENCES
(including Texts, Manuals, Specifications, Guidelines, Reports and Papers)

1. "Safety Policy Manual", Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

2. "A Guide on Applying for the Approval of Sewage Works", Municipal and Private Approvals Section, Environmental Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

3. "An Outline of the Information Required for the Preparation of a Hearing Brief Under Sections 43 and 44 of the Ontario Water Resources Act for the Environmental Assessment Board", Municipal and Private Approvals Section, Environmental Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

4. "Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment", Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario, November 1978.

5. "Guidelines for Land Use Surrounding Small and Medium Sized Sewage Treatment Plants" Environmental Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario, January 1978.

6. "Ice Piling on Lakeshores", Scientific Series 35, 1974 Environment Canada, Ottawa, Ontario.

7. Azan, K.M. and Boyko, B.I., "Identification of Problem Areas in Water Pollution Control Plants", Canada - Ontario Agreement Research Report No. 15, Environment Canada, Ottawa, Ontario.

8. James F. MacLaren Limited, "To Establish Viable Methods of Maintaining Waste Treatment Facility Efficiencies with Reference to Flow Variations", Canada - Ontario Agreement Research Report Nos. 11 and 55, Volumes I and II, Environment Canada, Ottawa, Ontario.

9. "Handbook for Sewer System Evaluation and Rehabilitation", Technical Report EPA 430/9-75-021, U.S. Environmental Protection Agency, Office of Water Program Operations, Washington, D.C. 20460, December 1975.

10. "Standard for Protected Steel Underground Tanks for Flammable and Combustible Liquids", Underwriter's Laboratory, Canada.

11. "Guidelines for the Provision of Equipment to Handle Emergency Conditions (Power Outages) in New Sewage Works in the Province of Ontario", Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario.

12. "Model Municipal Noise Control By-Law, Final Report", Ontario Ministry of the Environment, Ontario Government Bookstore, 880 Bay Street, Toronto, Ontario.

13. "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems and Storm Sewers", Municipal and Private Approvals Section, Environment Approvals Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario (May 1979).

14. Camp, Dress & McKee, Boston, Mass., "Wastewater Treatment Facilities for Sewered Small Communities", Process Design Manual EPA-625/1-77-009, U.S. Environmental Protection Agency (October 1977).

15. Camp, T.R. and Graber, S.D., "Dispersion Conduits", Journal of the Sanitary Engineering Division, ASCE (February 1968).

16. Davis, C.B. and Sorenson, K.E., "Handbook of Applied Hydraulics", McGraw-Hill Book Company, 330 West 42nd Street, New York (1969).

17. "Design and Construction of Sanitary and Storm Sewers" WPCP Manual of Practice No. 9 and ASCE Manual of Practice No. 37 (1970).

18. Chow, V.T., "Open-Channel Hydraulics", McGraw-Hill Book Company (1959).

19. "Wastewater Treatment Plant Design", ASCE Manual and Reports on Engineering Practice No. 36 and WPCF Manual of Practice No. 8 (1977).

20. "Sludge Treatment and Disposal", Process Design Manual EPA 625/1-79-011, U.S. Environmental Protection Agency (September 1979).

21. "Guidelines for Energy Conservation in the Design of Sewer Systems and Sewage Treatment Facilities in the Province of Ontario", Design and Equipment Section, Project Co-ordination Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario (October 1977).

22. Zarnet, G.D., "Energy Requirements for Conventional and Advanced Wastewater Treatment", Publication No. W47, Applied Sciences Section, Pollution Control Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario (October 1975).

23. Zarnet, G.D., "Optimization of Energy Allocation in Water and Wastewater Treatment Systems", Publication No. W65, Applied Sciences Section, Pollution Control Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario (March 1977).

24. Reid, Crowther and Partners Limited, "Conservation Measures in Waste Water Treatment Plants, A Feasibility Study", Ontario Ministry of Energy and Ontario Ministry of the Environment, Toronto, Ontario (October 1978).

25. "Energy Management Program (Energy Bus), Mobile Support Unit", Ontario Ministry of Industry and Tourism, Hearst Block, 900 Bay Street, Toronto, Ontario.

26. "Industrial Energy Conservation Handbooks (1-10)", Energy, Mines and Resources Canada, 580 Booth Street, Ottawa, Ontario.

27. "Canadian Code for Energy Conservation in New Buildings", National Research Council of Canada, Ottawa, Ontario.

28. "Chlorination of Wastewater", Manual of Practice No 4, Water Pollution Control Federation, 2626 Pennsylvania Avenue, N.W. Washington, D.C. (1976).

29. "Wastewater Filtration - Design Considerations", EPA Technology Transfer Seminar Publication EPA-625/4-7-4-007a.

30. "High Quality Effluents", Technology Transfer Seminar Sponsored Under Canada-Ontario Agreement on Great Lakes Water Quality, Toronto, December 2 and 3, 1975.

31. "Summary Report on the Phosphorus Removal Program" Municipal and Private Section, Pollution Control Branch, Ontario Ministry of the Environment, 135 St. Clair Avenue West, Toronto, Ontario (April 1976).

32. Archer, J.D., Ontario Ministry of the Environment, "Summary Report on Phosphorus Removal", Canada-Ontario Agreement Research Report No. 83, Environment Canada, Ottawa, Ontario.

33. Graham, H.J. and Hunsinger, R.B., Ontario Ministry of the Environment, "Phosphorus Removal in Seasonal Retention Lagoons by Batch Chemical Precipitation" Canada-Ontario Agreement Research Report No. 13, Environment Canada, Ottawa, Ontario.

34. Graham, H.J. and Hunsinger, R.B., Ontario Ministry of the Environment, "Phosphorus Reduction from Continuous Overflow Lagoons by Addition of Coagulants to Influent Sewage", Canada-Ontario Agreement Research Report No. 65, Environment Canada, Ottawa, Ontario.

35. "Phosphorus Removal Design Seminar", Canada-Ontario Agreement, Toronto (May 28-29, 1973).

36. "Process Design Manual for Phosphorus Removal", Technical Report EPA 625/1-76-001a, U.S. Environmental Protection Agency, Technology Transfer (April 1976).

37. Pollutech Pollution Advisory Services Limited, Oakville, Ontario, "Chemical Dosage Control for Phosphorus Removal", Canada-Ontario Agreement Research Report No. 4, Environment Canada, Ottawa, Ontario.

38. Boyko, B.I. and Rupke, J.W.G., Ontario Ministry of the Environment, "Phosphorus Removal Within Existing Wastewater Treatment Facilities", Canada-Ontario Agreement Research Report No. 44, Environment Canada, Ottawa, Ontario.

39. Ongerth, J.E., Brown and Caldwell Incorporated, "Evaluation of Flow Equalization in Municipal Waste Treatment", EPA-600/2-79-096, Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency (May 1979).

40. Sax, N.I., "Dangerous Properties of Industrial Materials".

41. "Hazardous Chemicals Data", NFPA No. 49, National Fire Protection Association, 470 Atlantic Avenue, Boston, MA. (1975).

42. "Guidelines for Environmental Protection Measures at Chemical Storage Facilities", Waste Management Branch, Ontario Ministry of the Environment (October 1978).

43. "Installation Code for Digester Gas Systems", CAN1-B105-M81, Canadian Gas Association, 55 Scarsdale Road, Don Mills, Ontario.

44. Campbell, H.W., Rush, R.J. and Tew, R., "Sludge Dewatering Design Manual", Canada-Ontario Agreement Research Report No. 72, Environment Canada, Ottawa, Ontario.

45. "Sludge Utilization and Disposal", Seminar Notes, The Land Disposal of Sludge Subcommittee, Canada-Ontario Agreement, Toronto (February 20, 21, 1978).

46. "Sludge Handling and Disposal Seminar", Canada-Ontario Agreement Conference Proceedings No. 2, Toronto (September 18, 19, 1974).

47. "Process Design Manual for Sludge Treatment and Disposal", Technical Report EPA 625/1-79-011, U.S. Environmental Protection Agency, Office of Research and Development (September 1979).

48. "Sludge Treatment and Disposal, Sludge Treatment", Volume I, Technical Report EPA 625/4-78-012, U.S. Environmental Protection Agency, Environmental Research Information Center, Cincinnati, Ohio (October 1978).

49. "Instrumentation in Wastewater Treatment Plants", Manual of Practice No. 21, Water Pollution Control Federation, 2626 Pennsylvania Avenue, N.W., Washington, D.C. (1978).

50. "Operation of Wastewater Treatment Plant", Manual of Practice No. 11, Water Pollution Control Federation, Washington, D.C. (1976).

51. Rush R.J., Rush Engineering Services Limited and Stickney, A.R., Wastewater Technology Centre, Environmental Protection Services, Environment Canada, "Natural Freeze-Thaw Sewage Sludge Conditioning and Dewatering" Report No. EPS 4-WP-79-1 (January 1979).

52. Penman, A., and Van Es, D.W., City of Winnipeg, "Winnipeg Freezes Sludge, Slashes Disposal Costs 10 Fold", Civil Engineering, American Society of Civil Engineers (November 1973).

53. Fielding, M.B., "The Use of Deep Lagoons for Thickening and Drying of Digested Sludge", Division of Research, The Ontario Water Resources Commission, Publication No. 12 (June 1966).

54. Andrews, R.H., "Sludge Treatment and disposal", Division of Research, The Ontario Water Resources Commission, Publication No. 23 (January 1967).

55. Ehlert, N.G., "Site Considerations and Pilot Studies for Land Disposal of Treated Sewage Effluent", Publication No. W56, Applied Sciences Section, Pollution Control Branch, Ontario Ministry of the Environment (December 1975).

56. Ayers, H.D. and Spencer, V.I.D., University of Guelph and Ontario Ministry of Agriculture and Food, "Irrigation Practices for Ontario", AGDEX 560/753, Ontario Ministry of Agriculture and Food, Parliament Buildings, Toronto.

57. "Spray Irrigation of Treated Municipal Wastewater", Economics and Technical Review Report EPS 3-PR-76-1, Fisheries and the Environment, Environmental Protection Service, Ottawa, Ontario.

58. Stevenson, C.D., "Interim Guide for Land Application of Treated Sewage Effluent", Information Series No. 114, Department of Scientific and Industrial Research, Wellington, New Zealand (1976).

59. "Health Aspects of Sewage Effluent Irrigation", Pollution Control Branch, British Columbia Water Resources Service, Department of Lands, Forests, and Water Resources, Parliament Buildings, Victoria, British Columbia (April 1975).

60. Sopper, W.E., "Crop Selection and Management Alternatives - Perennials", Conference Proceedings on Re-cycling Municipal Sludges and Effluents on Land at Champaign, Illinois (July 1973).

61. "Class Environmental Assessment for expansion or upgrading of an existing sewage system", Project co-ordination Branch, Ministry of the Environment, 40 St. Clair Avenue West, Toronto, Ontario.

62. "Units of Expression for Wastewater Management", Manual of Practice No. 6, Water Pollution Control Federation, 2626 Pennsylvania Avenue, N.W. Washington, D.C. 20037.

63. Addendum to "MOE Class Environmental Assessment Document Expansion or Upgrading of an Existing Sewage /Water System", Environment, 40 St. Clair Avenue West, Toronto, Ontario.

64. "Guidelines for Sewage Sludge Utilization on Agricultural Lands", Ontario Ministry of Agriculture and Food and Ontario Ministry of the Environment.

65. "Wastewater Stabilization Pond Linings" MCD-54, EPA, General Services Administration (8FFS), Centralized Mailing List Services, Bldg. 41, Denver Rederal Center, Denver, Colorado 80225.

66. Houck, Daniel H. and Boon, Arthur, G., "Survey and Evaluation of Fine Bubble Dome Diffuser Aeration Equipment" Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio, 45268.

LEGISLATION

201. "The Ontario Water Resources Act", Government of Ontario, Queen's Park, Toronto, Ontario.

202. "The Environmental Assessment Act", Government of Ontario, Queen's Park, Toronto, Ontario.

203. "The Environmental Protection Act", Government of Ontario, Queen's Park, Toronto, Ontario.

204. "National Fire Code of Canada" Associate Committee on the National Fire Code, National Research Council of Canada, Ottawa, Ontario.

205. "The Occupational Health and Safety Act, and Regulations for Construction Projects", Government of Ontario, Queen's Park, Toronto, Ontario.

206. "The Occupational Health and Safety Act, and Regulations for Industrial Establishments", Government of Ontario, Queen's Park, Toronto, Ontario.

207. "Consolidated Hearings Act", Government of Ontario, Queen's Park, Toronto, Ontario.

MINISTRY OF THE ENVIRONMENT
EQUIPMENT SPECIFICATIONS AND GUIDELINES

300 MOE Standard Equipment Procurement Specifications

301 Factory Built Underground Pumping Stations #1
Issue No. 5 of April, 1973

302 Diesel Generator Set #2
Issue No. 7 of June, 1981

303 Circular Clarifiers #7
Issue No. 1 of September, 1983

304 Magnetic Flowmeters #9
Issue No. 1 of July, 1975

NOTE: Many of the following draft specifications have been prepared by various consultants and may be considered as the state of the art at the present time. They are being continuously updated as improved specifications become available.

400 MOE DRAFT EQUIPMENT PROCUREMENT SPECIFICATIONS

(A) PUMPS

401 Submersible Sewage Pumps #3
Draft of July, 1974

402 Dry Pit Sewage Pumps #4
Draft of January, 1975

403 Progressive Cavity Pumps
Draft of February, 1978

404 Plunger Pumps
Draft of May, 1975

405 Vertical Turbine Lineshaft Pumps #16
Draft of June, 1977

406 Vertical Turbine Pumps - Submersible Motor
Draft of July, 1973

407 Vertical Split Case Water Pumps
Draft of February, 1976

408 Horizontal Split Case Water Pumps
Draft of October, 1977

(B) CHEMICALS AND CHLORINATION

409 Polymer Feeding Systems
Draft of November, 1977

410 Chemical Mixers
Draft No. 1 of February, 1976

411 Chlorination Equipment
Draft No. 1 of April, 1975

(C) DIGESTER

412 Digester Steel Covers
Draft of June, 1975

413 Digester Gas Mixing Equipment
Draft of June, 1983

414 Digester Heat Exchangers
Draft of October, 1975

(D) MISCELLANEOUS

415 Sluice Gates and Gate Operators
Draft No. 1 of February, 1976

416 Outdoor Chemical Storage Tanks
Draft of June, 1982

500 MOE GUIDELINE REQUIREMENTS FOR EQUIPMENT PROCUREMENT

(A) AERATION

501 Positive Displacement Blowers #6
Draft of January, 1980

502 Centrifugal Blowers
Draft of June, 1982

503 Diffused Air-Coarse and Fine Bubble
Draft of September, 1980

504 Mechanical Aerators #5
Draft of July, 1979

(B) BOILERS

505 Fire Tube Boilers
Draft of July, 1975

506 Water Tube Boilers
Draft of October, 1975

507 Coil Tube Boilers
Draft of November, 1977

(C) INFLUENT

508 Mechanically Cleaned Bar Screens
(Hydraulic Activated arm and chain types)
Draft of October, 1975

509 Mechanically Cleaned Bar Screens
(Cable operated type)
(Chain operated type)
Draft of July, 1980 (#21)

510 Grit Removal Equipment
(Detrittor type)
Draft of October, 1975

511 Grit Removal Equipment
(Air degritter type)
Draft of September, 1978

512 Comminutor #19 (Drum type)
Draft of April, 1981

513 Traveling Screens for WTP
Draft of July, 1977

(D) SLUDGE TREATMENT

514 Vacuum Filter
Draft of November, 1978

515 Flotation Thickener
Draft of November, 1978

516 Centrifuge - Solid Bowl and Disc Nozzle
Draft of November, 1978

517 Sludge Belt Filter
Draft of February, 1983

(E) CLARIFIERS

518 Bridge Type Clarifiers
Draft of June, 1978

519 Scum Collectors
Draft of June, 1978

(F) MISCELLANEOUS

520 Large Dry Pit Sewage Pumps with Variable Speed Drives
Draft of June, 1982

521 Screw Sewage Pumps
Draft of April, 1977

522 Chemical Metering Pumps
Draft of 1974

523 Diesel Engine for Vertical Turbine Pumps
Draft of January, 1973

524 Telephone Digital Dialers for Alarm systems
Draft of February, 1981

525 Automatic Effluent Filter
Draft of June, 1982

600 MOE GUIDELINES REQUIREMENTS FOR EQUIPMENT INSTALLATION

601 Chlorination Rooms and Auxiliary Equipment
Draft of 1977

602 Sewage Treatment Plant (Package-Type)
Draft of July, 1977